AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

335.
(Vol. XV.—June, 1886.)

ON SPECIFICATIONS FOR STRENGTH OF IRON BRIDGES.

By JOSEPH M. WILSON, M. Am. Soc. C. E.
READ AT THE ANNUAL CONVENTION, JUNE 25TH, 1885.

WITH DISCUSSION.

It has been within the province of the author, in his connection with the Pennsylvania Railroad Company, to prepare the specifications on which plans for all iron bridges for that line should be designed.

A long experience not only in designing, but also in the supervision and periodical inspection of bridges in actual use on the various lines of the company, has given the author opportunity of observing their action in service from year to year, and of noting any suggestions which might be furnished as to modifications or improvements in design. This experience has been of considerable value in forming the author's judgment on many points, and the results have necessitated more or less modification, from time to time, of the specifications under which work was designed. The great increase, however, in the weight of motive power and train service, and in the speed of running, has required of late years some very marked changes.

The specifications which are herewith submitted (see Appendix) are the result of theory combined with practice, and this paper is offered in

the hope that it may contribute something to the profession. These specifications are now standard for the Pennsylvania Railroad Company.

To meet the question of the continual increase in the weights of engines, necessitating constant changes in the assumed loading of bridges, it was decided several years ago to adopt for computation certain typical forms outlined under the author's direction, which, while covering the then existing service, would, it was believed, represent the advance for a longer period, and yet be reached in the near future. The loads from these typical consolidation and passenger engines are given in the specification, and results have already more than justified their adoption, as although no engines have as yet been built on the Pennsylvania Railroad with these precise weights, yet another type has actually developed in construction (P. R. R., Class M) which, in particular cases produces even heavier loading than that given by the typical engines, and indicates that the limit of increase has been by no means yet reached.

The specification starts out with the well-known requirement that the structure must be wholly of wrought-iron, and then follows with the live loads as represented by the typical and "Class M" engines, diagrams being given for each. The maximum, as given by either of these methods of loading, is required to be used, and in calculating web-members of trusses and girders, the cross-girder load under the drivers is to be considered as the head of the train, the load under the preceding cross-girder being neglected. This is on the side of safety.

The dead load to be carried is then stated, and its mode of distribution is given for the loaded and unloaded chords. The points from which general dimensions are to be taken for calculation are designated, and with the conditions now given the designer computes the stresses by one of the recognized methods.

The maximum and minimum stresses in compression and tension being found for the various members of the structure, depending upon the type of bridge selected by the designer, the specification then states the formulas according to which the permissible working stresses in these pieces are to be determined. These formulas are modified from those of Launhardt and Weyrauch as given by Weyrauch. ("Strength and Calculation of Dimensions of Iron and Steel Constructions." New York, D. Van Nostrand, 1877.)

Launhardt's formula requires:

For pieces subject to one kind of stress only (tension or compression):

$$a=u\left(1+\frac{t-u}{u}\ \frac{\text{Min. } B}{\text{Max. } B}\right)$$

Weyrauch determined (for pieces subject to stresses acting in opposite directions):

$$a=u\left(1-\frac{u-s}{u}\ \frac{\text{Max. } B'}{\text{Max. } B}\right)$$

where

a = ultimate working strength per unit of section under assumed conditions of loading.

u = ultimate strength per unit of section for any number of repetitions of load.

t = ultimate strength per unit of section for a single static load.

s = ultimate vibrating strength per unit of section when the opposite stresses are equal.

Max. B = the greatest stress upon the member, whether of tension or compression.

Min. B = the least stress of the same kind.

Max. B' = the greatest stress in the opposite sense.

Now, assuming with Weyrauch that $t=1\frac{1}{2}u$ and $s=\frac{1}{2}u$, there results:

$$a=u\left(1+\frac{\text{Min. } B}{2\,\text{Max. } B}\right)$$

and

$$a=u\left(1-\frac{\text{Max. } B'}{2\,\text{Max. } B}\right)$$

These formulas, with the proper substitution for u, depending upon the kind and quality of material used, will give the ultimate strength for the varying conditions of live and dead load quite satisfactorily, but they do not take account of the question of impact, etc., such as would be caused by the weight and motion of the train, irregularity of track, and the vertical pressure due to the inclination of the connecting rod of the engine, as well as centrifugal force developed from unbalanced weight in the wheels or other revolving parts. This question has been more or less discussed by engineers for a long time. Mr. B. Baker, C. E., in his work on "Long and Short Span Railway Bridges" (London, 1873), treats of the subject, and to provide for this, as well as the varying

conditions of live and dead loads, suggests the addition of a percentage to the usual working stress for bridges of short spans. This method of meeting the question has been adopted by quite a number of engineers. The following list, showing the percentages of increase given to static stresses by various designers, has been compiled from a number of examples of specifications to which the author has had access.

Edge Moor Iron Company :

Span in feet.....	20	30	40	50	60	70	80
Per cent........	30	25	20	15	12	10	8

Keystone Bridge Company, for truss members only :

Span in feet.....	30	45	60	75	90	105	120	135	150
Per cent........	20	15	12	10	8	6	4	2	0

For stringers, floor-beams and counter-ties, 25 per cent.; floor-beam hangers, 100 per cent.; riveted connections of stringers and floor-beams, 50 per cent.; middle-ties and posts, 10 per cent.

Mr. G. Bouscaren, M. Am. Soc. C. E.:

Span in feet.................	30	45	60	75
Per cent....................	25	20	15	10

Floor-beam hangers, 50 per cent.

Mr. Mace Moulton, M. Am. Soc. C. E., founded on B. Baker's deductions :

Span in feet....	10	20	30	45	60	80	100
Per cent.......	40	35	30	25	20	15	10

Mr. C. Shaler Smith, M. Am. Soc. C. E.:

Middle-ties, 20 per cent.; counters, 30 per cent.; end suspenders, 20 per cent.; long suspenders, 20 per cent.; floor-beam hangers, 50 per cent.; track-girders, 30 per cent.; floor-beams, 20 per cent.

Henderson bridge specifications :

Span in feet.............	30 to 45	45 to 60	60 to 75	75 to 120
Per cent. compression..	20	15	10	—
" tension.......	40	25	20	15

Floor-beam hangers and track-girder connections, 100 per cent.; vertical suspenders, 50 per cent.; track-girders, floor-beams, counters, trestle-posts and compression members of less than 30 feet span, 25 per cent.

These authorities adopt only one value of static stress for all classes of material, and in all cases, except for the Henderson bridge, 10 000 pounds per square inch is used for tension, and 8 000 pounds per square inch

for compression. The Henderson bridge specifications required 10 000 pounds per square inch for tension, except in track-girders and floor-beams, for which 8 000 pounds was used. For compression, 8 000 pounds was adopted, as given in the other cases.

Prof. William Cain, A. M., C. E. ("Maximum Stresses in Framed Bridges," New York, 1878), takes a more rational method of solving the problem. He assumes that as impact is due to and increases with the action of the live load on a member, its effect will vary inversely with $\frac{\text{Min. } B}{\text{Max. } B}$ and he writes empirically for the formula of Launhardt, $b=\frac{u}{n}\left(1+\frac{\text{Min. } B}{\text{Max. } B}\right)$, n being a factor of safety; and he takes the value of $\frac{u}{n}$ for all iron at 7 500 pounds per square inch. Here the value of b is the permissible working stress per unit of section. This is equivalent to making the value of $t=2\,u$ in Launhardt's formula previously given. Adopting this relationship between t and u, and still retaining that of $s=\frac{1}{2}\,u$, the original formulas become

$$a=u\left(1+\frac{\text{Min. } B}{\text{Max. } B}\right)\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

$$a=u\left(1-\frac{\text{Max. } B'}{2\,\text{Max. } B}\right)\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

These formulas, it must be remembered, are for ultimate strength. If for u, working values are assumed instead of ultimate, the value of a becomes permissible working stress per unit of section, and adopting this arrangement, the author believes that the results of practical experience are most rationally met.

The value of u is taken by the author for double-rolled iron in tension (links or rods), at 7 500 pounds per square inch; for rolled iron in tension (plates or shapes), at 7 000 pounds per square inch; and for rolled iron in compression, at 6 500 pounds per square inch.

For Min. $B=$ Max. B, $a=2\,u=t=15\,000$ pounds, corresponding to all dead load, and for Min. $B=0$, $a=u=7\,500$, corresponding to all live load.

Plate XXXIV, accompanying this paper, gives by diagram the value of a for various values of $\frac{\text{Min. } B}{\text{Max. } B}$ according to the formulas of the specification.

For high test iron in tension, the line $A\,A$ should be used for the

first case, pieces subjected to one kind of stress only, and the line *A D* for the second case, pieces subject to stresses acting in opposite directions. For ordinary iron in tension, use the line *B B* for first case, and *B E* for second case. For ordinary iron in compression, use the lines *C C* and *C F*.

On the same diagram there will be found plotted the values of a resulting from the various methods of percentage as used by Edge Moor Iron Company, Keystone Bridge Company, etc., previously quoted, and although the lines shown may be somewhat in error, as the author was obliged to assume his own values of loads for the various spans, still the results are valuable as a matter of comparison with those obtained by the formulas.

The permissible stress a being found, that for members in compression must be reduced in proportion to the ratio of the length to the least radius of gyration of the section.

The author has for many years used Rankine's formulas for this purpose. Late experiments, however, made on a large scale, and the investigations of a number of authorities on the subject, do not altogether agree with the results of these formulas.

During the years 1875 to 1879, inclusive, Mr. G. Bouscaren, M. Am. Soc. C. E., made a series of experiments on the strength of wrought-iron columns, the results of which will be found in a paper contributed by him to the Society (Trans. Am. Soc. C. E., Vol. IX, p. 447). From these results it would appear that between Rankine's and Gordon's formulas, the former is the more accurate of the two: "that iron of the highest modulus does not necessarily make the strongest column; that it is essential in built columns to thoroughly fasten together the several parts or segments that they may act in a solidary manner as one continuous metallic body, and to guard not only against lateral flexure or buckling of each part, but also against longitudinal relative motion; that a due regard to the economy of material demands that the thickness of the metal, and in built columns the spacing of the rivets, be so proportioned that the column shall fail by flexure as a whole before any local buckling or flexure shall take place;" that Rankine's formula with the constant $\frac{1}{3000}$ for columns with flat ends, assumed at double its value for columns hinged on pins at both ends, "is practically correct."

The tables accompanying Mr. Bouscaren's paper give the results of his experiments on various columns of different kinds, forms of cross-sections, etc., and made by different manufacturers.

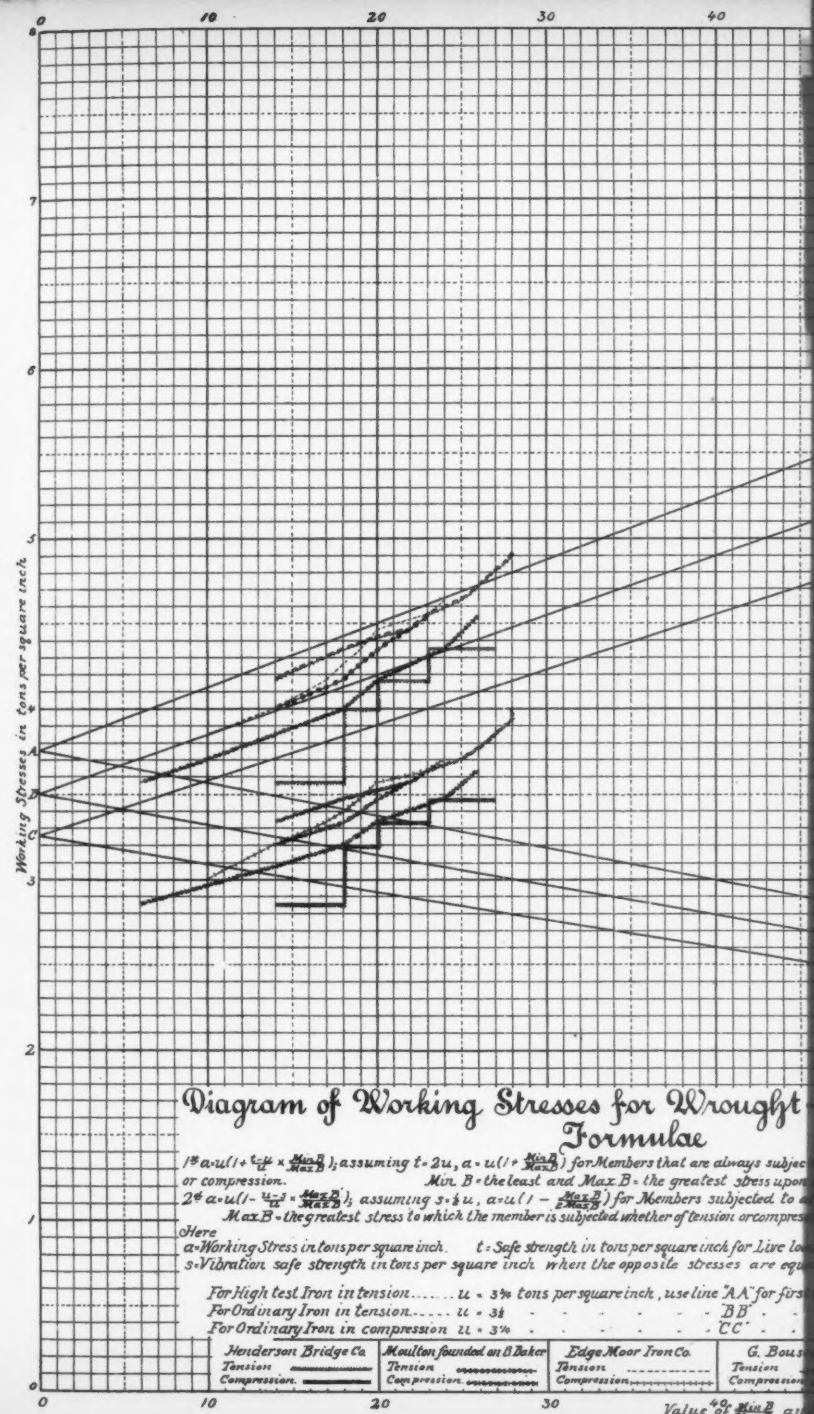
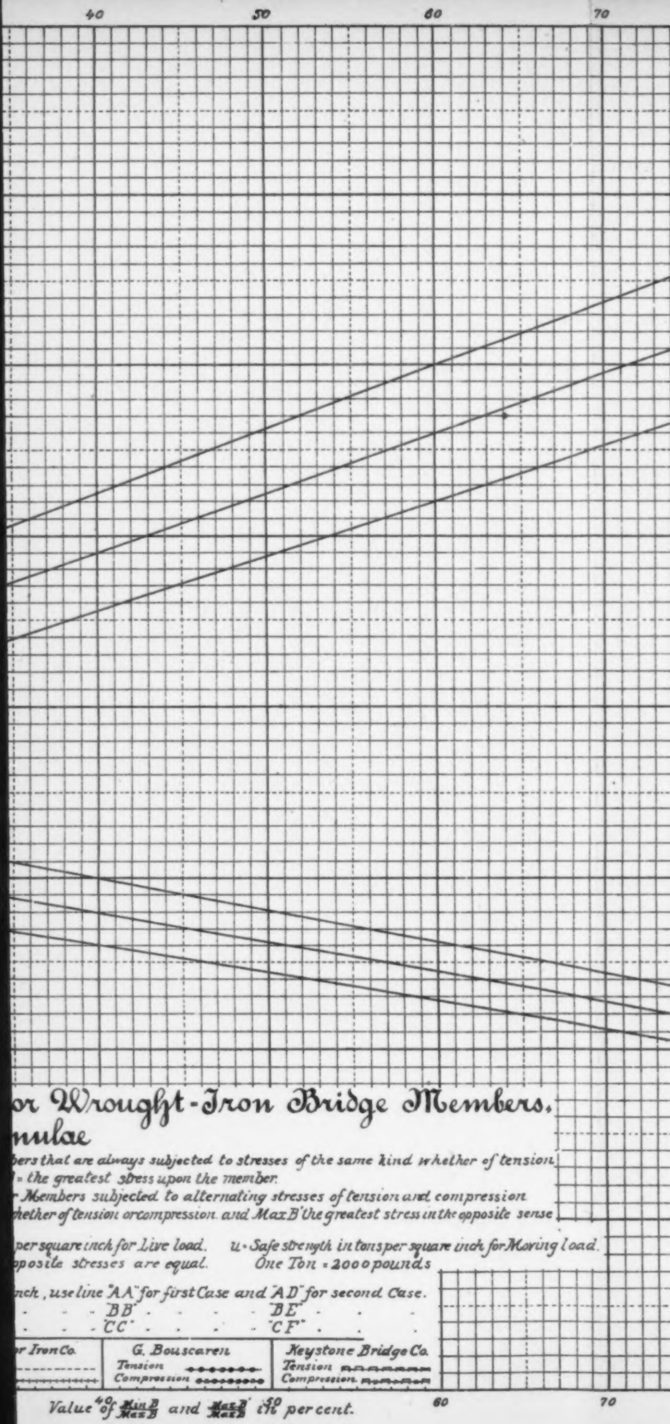


Diagram of Working Stresses for Wrought
Formulae
1st $a = u(1 + \frac{t-u}{u} \times \frac{Min B}{Max B})$; assuming $t = 2u$, $a = u(1 + \frac{Min B}{Max B})$ for Members that are always subjec
or compression.
Min. B = the least and Max. B = the greatest stress upon
2d $a = u(1 - \frac{u-s}{u} \times \frac{Max B'}{Max B})$; assuming $s = \frac{1}{2}u$, $a = u(1 - \frac{Max B'}{2 Max B})$ for Members subjected to a
Max B = the greatest stress to which the member is subjected whether of tension or compres
Here
a = Working Stress in tons per square inch.
t = Safe strength in tons per square inch for Live lo
s = Vibration safe strength in tons per square inch when the opposite stresses are equ
For High test Iron in tension .... u = 3¾ tons per square inch, use line "AA" for firs
For Ordinary Iron in tension...... u = 3½ - - - - - - "BB"
For Ordinary Iron in compression u = 3¼ - - - - - - "CC"
Henderson Bridge Co.
Tension
Compression.
Moulton founded on B. Baker
Tension
Compression
Edge Moor Iron Co.
Tension
Compression
G. Bous
Tension
Compression
Working Stresses in tons per square inch
Value of $\frac{Min B}{Max B}$
A
B
C
0
10
20
30
40
1
2
3
4
5
6
7
8


40
50
60
70
or Wrought-Iron Bridge Members.
nulae
bers that are always subjected to stresses of the same kind whether of tension
= the greatest stress upon the member.
r Members subjected to alternating stresses of tension and compression
hether of tension or compression. and Max B' the greatest stress in the opposite sense
per square inch for Live load.
u = Safe strength in tons per square inch for Moving load.
pposite stresses are equal.
One Ton = 2000 pounds
nch, use line "AA" for first Case and "AD" for second Case.
"BB" "BE"
"CC" "CF"
r Iron Co.
G. Bouscaren
Tension
Compression
Keystone Bridge Co.
Tension
Compression
Value of Min B / Max B and Max B' / Max B in per cent.
40
50
60
70

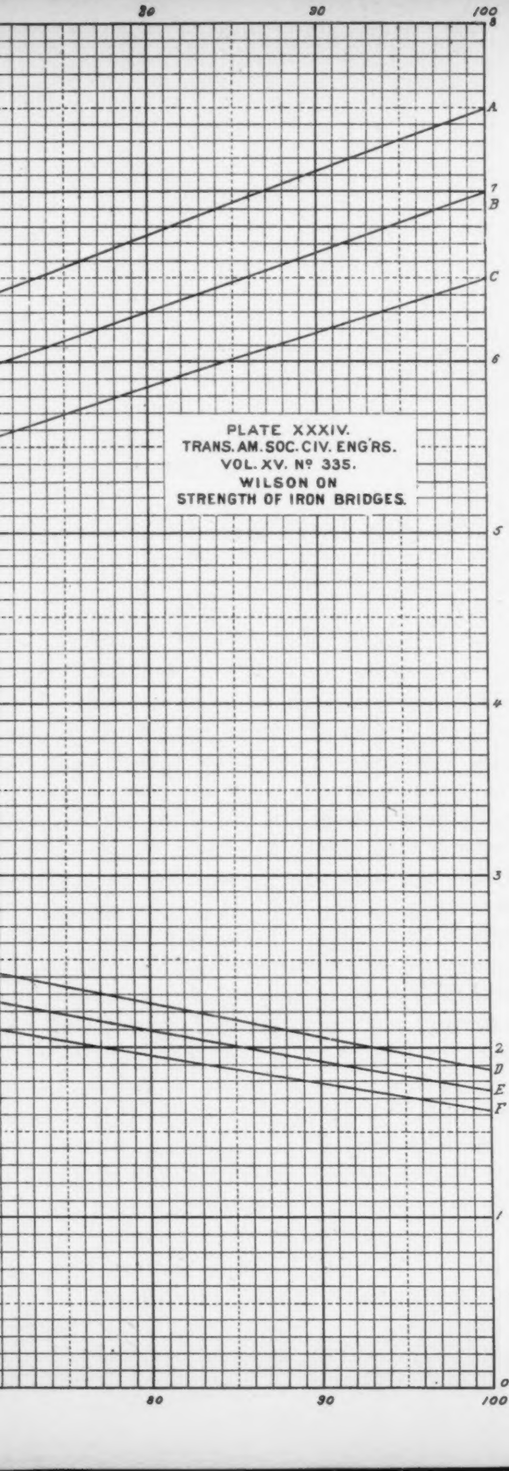


PLATE XXXIV.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. Nº 335.
WILSON ON
STRENGTH OF IRON BRIDGES.
80
90
100
A
B
C
D
E
F
0
1
2
3
4
5
6
7
8

In November, 1879, Messrs. Clarke, Reeves & Co., Mems. Am. Soc. C. E., had a series of experiments made upon a number of Phœnix wrought-iron columns at Watertown Arsenal, Mass., and the results of their work were furnished to the Society in 1882 (Trans. Am. Soc. C. E., Vol. XI). These experiments on breaking strength, elastic limit, etc., embraced twenty columns of 8.04 inches diameter, formed of four rolled arc segments having flanged riveted connections, and lengths varying from 8 inches to 28 feet, the sectional area of each being slightly in excess of 12 square inches; also two columns 11.8 inches in diameter and 18.3 square inches sectional area, formed of six segments and having lengths of 8 feet 9.5 inches and 25 feet 2.65 inches respectively. The results demonstrated the inaccuracy of Gordon's formulas for hollow columns, as might be supposed, and indicated the advisability of using separate formulas for long and short columns, as suggested by Hodgkinson some years ago.

Rankine's formulas corresponded much more closely with the experiments, indicating, however, that the value of $\frac{1}{36000}$ for a was too large, $\frac{1}{100000}$ being better.

In the discussion on this paper, Mr. Theodore Cooper, M. Am. Soc. C. E., proposes a series of formulas for columns of different forms of section and for the various conditions of square, pin or round-ended. He thinks that the allowed working strain should be a proportion of the elastic limit of the material, not of the rupturing or crippling-point. Instead of computing the crippling strength of a column, and taking, say one-fourth, one-fifth or one-sixth of the result as the allowed strain, he prefers changing the formulas so as to give directly the allowed strain, and suggests modifications for this purpose.

Mr. James Christie, M. Am. Soc. C. E., read before the Society, in 1883, a very complete paper on experiments made by him at the Pencoyd Iron-works for the purpose of determining the comparative resistance to compression of long and short struts of rolled angle, tee-beam and channel sections. This paper forms a valuable contribution to available data on the practical resistance of columns.

On Plates XXXV and XXXVI accompanying, will be found the actual ultimate resistance, as shown by the principal experiments reported in these various papers, Plate XXXV being for fixed, and Plate XXXVI for pinned ends. On Plate XXXVI are shown also the results of four experiments made under direction of the author at

Watertown Arsenal several years ago. They are designated on the plate Penna. R. R. Two of these were made on columns built of channels with a middle web of plate secured by angles, and two upon columns of channels having their flanges connected by cross-plates and lattice-bars. All the specimens were of the same size and length of channels, and had pins at both ends, proper shoes being made to fit against the plates of the testing machine and to receive the pins. The columns were tested with the pins vertical, and no effort was made to counterbalance the weight of the specimens.

The results of the various experiments are plotted in the diagram by using for abscissæ the values of the length divided by the least radius of gyration or $\frac{l}{r}$, and for ordinates the breaking stress per unit of section. The scales adopted have been so taken as to avoid crowding the results together, and to enable a better comparison to be made between them. Very often in such diagrams the vertical scale is taken too small.

On these same plates will be found also plotted the lines of ultimate resistance for columns as by the formulas of Rankine, Cooper and Bouscaren-Rankine, the latter being modifications of Rankine's formulas as proposed by Mr. Bouscaren ("Report on the Progress of Work and Cost of Completing and Maintaining the Cincinnati Southern Railway," G. Bouscaren, January 1, 1880, p. 20). Mr. Bouscaren takes the ultimate resistance of column per square inch $= R = \dfrac{f}{1 + \dfrac{l^2}{36\,000\, r^2}}$ for flat ends, and $R = \dfrac{f}{1 + \dfrac{l^2}{18\,000 r^2}}$ for pin ends, and the value of f being assumed at 45 000 for Phœnix post, 38 600 for square post closed on all sides, or open and latticed on two opposite sides, and 36 500 for H post. The factor of safety he takes at one-fifth. There will also be found plotted Mr. Bouscaren's proposed modification of Rankine, using $f = 38\,000$, and $a = \frac{1}{100\,000}$ (Trans. Am. Soc. C. E., Vol. XI, p. 65), and Dr. Winkler's modification of Rankine (Proc. Inst. C. E., Vol. LIII, p. 302), where $f = 40\,000$; $a = 0.000\,044$. By examination of these diagrams the variation between the results of the formulas and the actual experiments may readily be observed.

After considerable consideration of the subject, the author has adopt-

late
nels
nns
ice-
els,
the
nns
ter-

by
s of

sec-

the
een
too

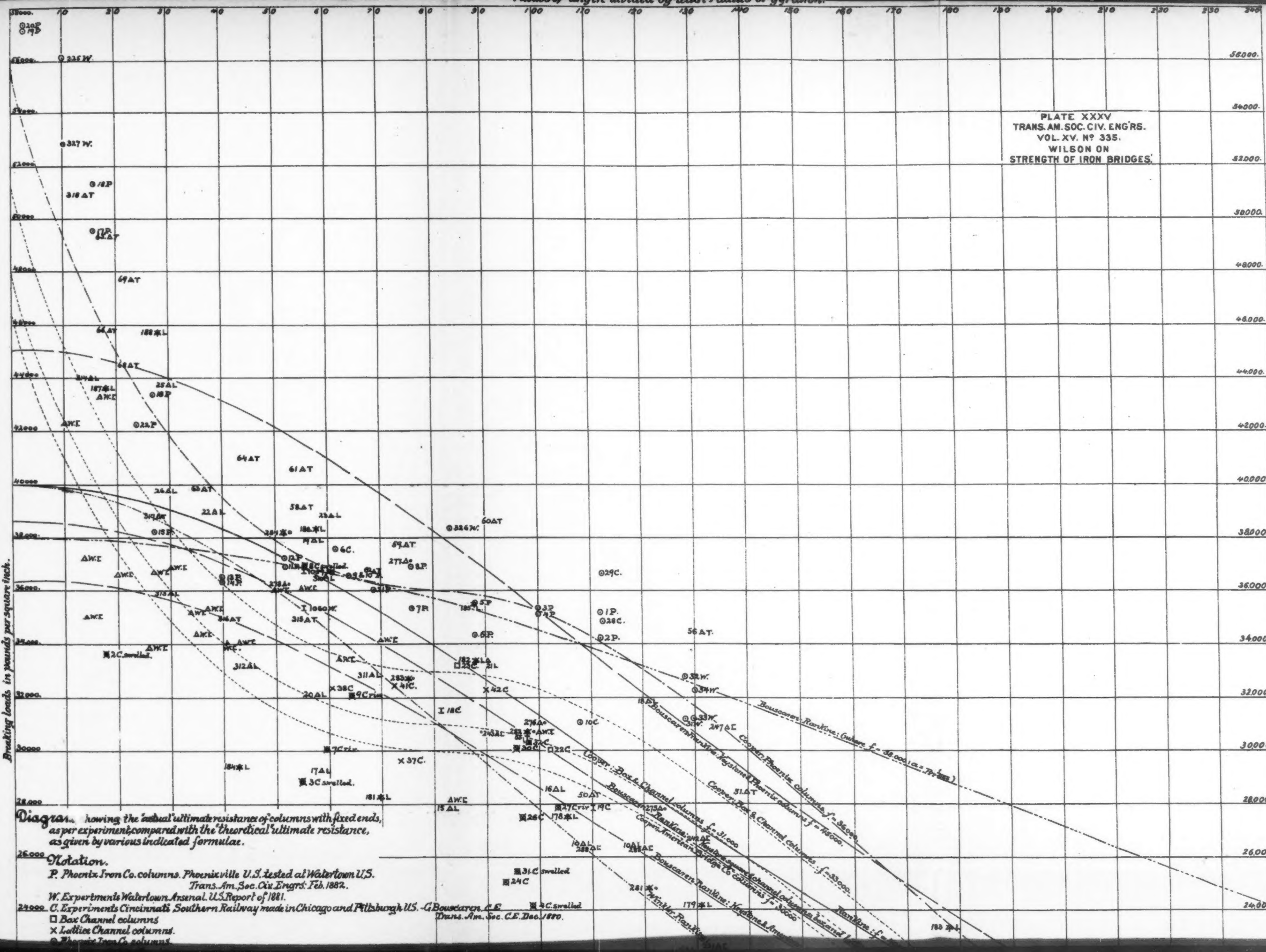
nate
ous-
s as
Cost
G.
re-

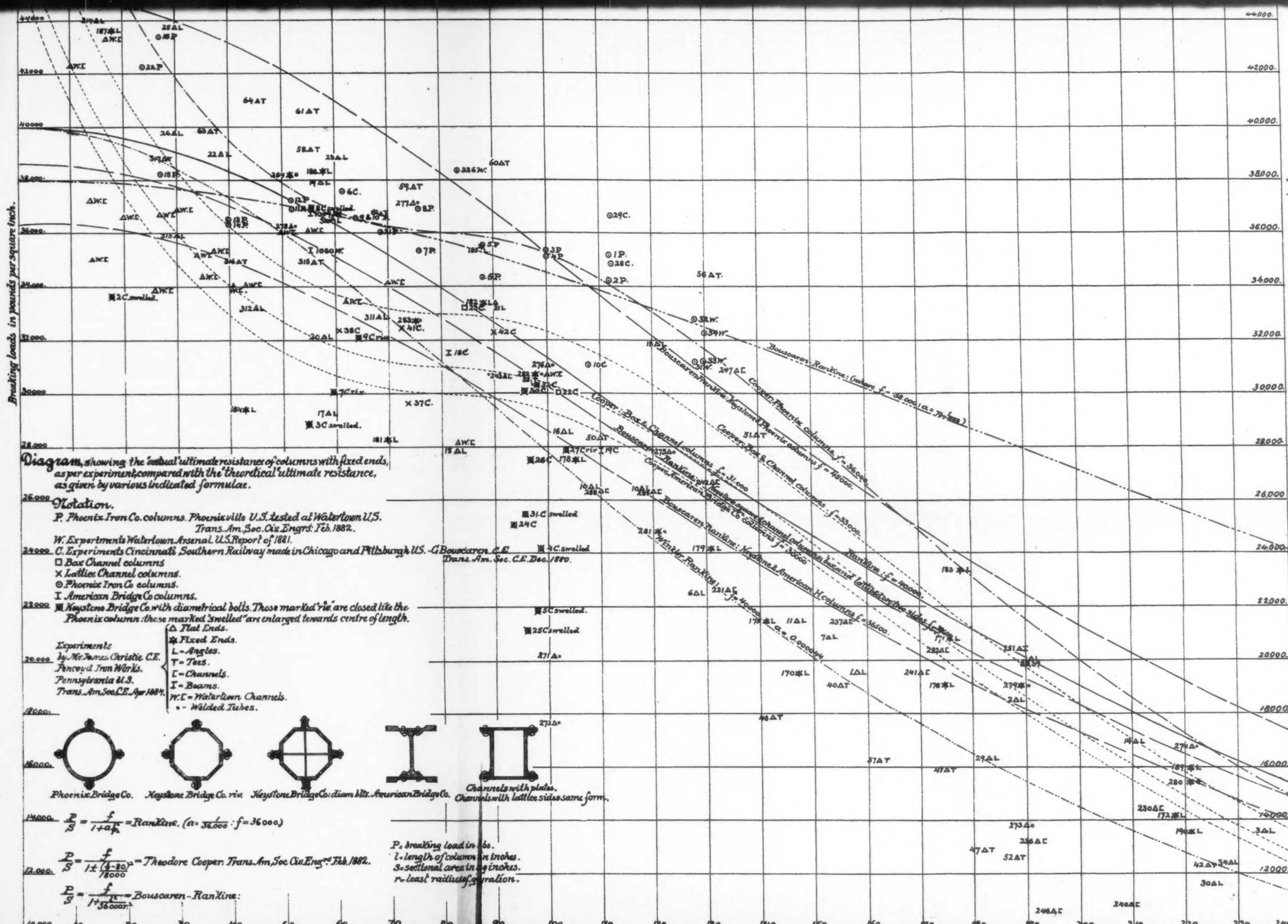
nds,

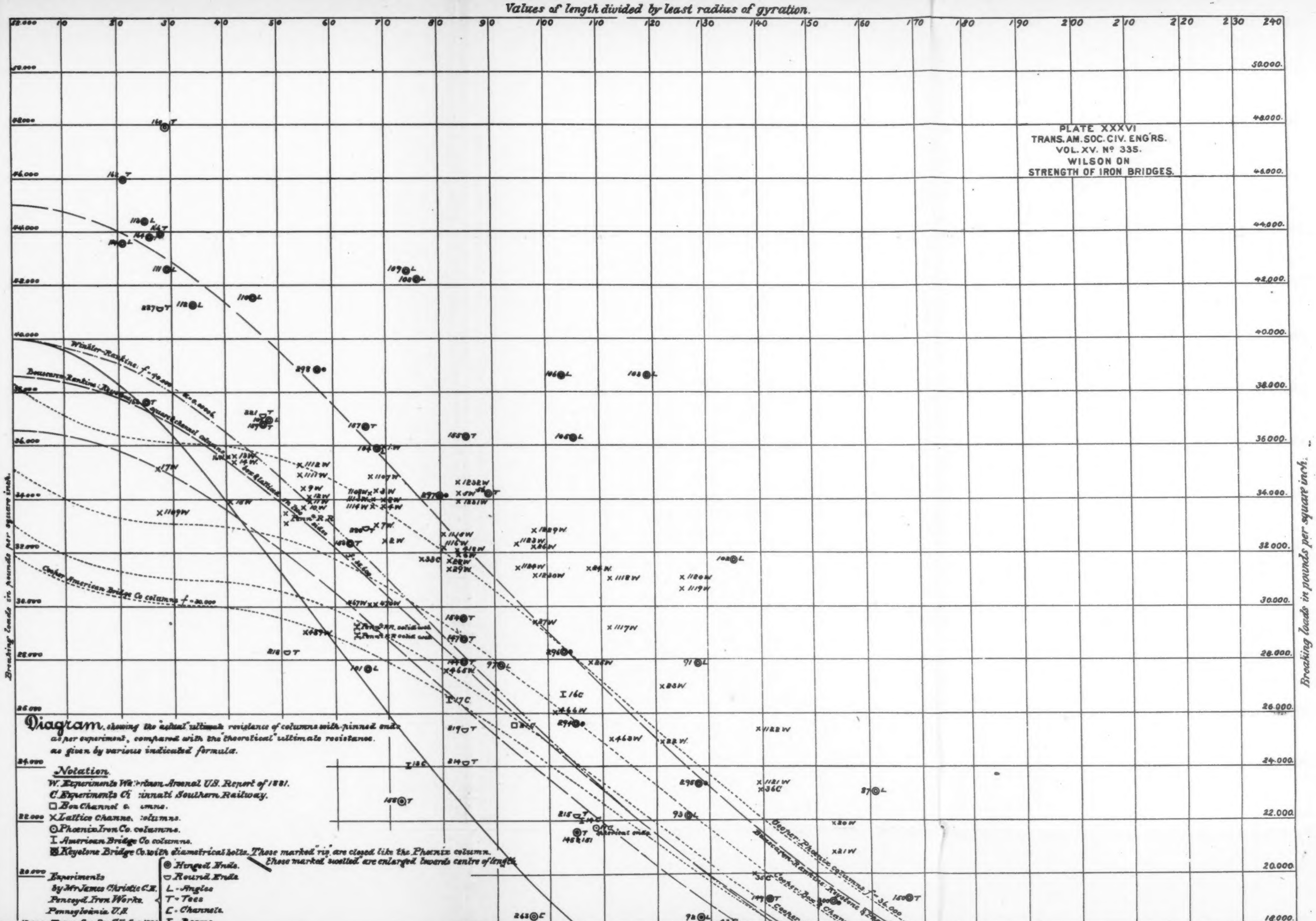
d at

pen
r of
Mr.
and
ler's
here
ria-
may

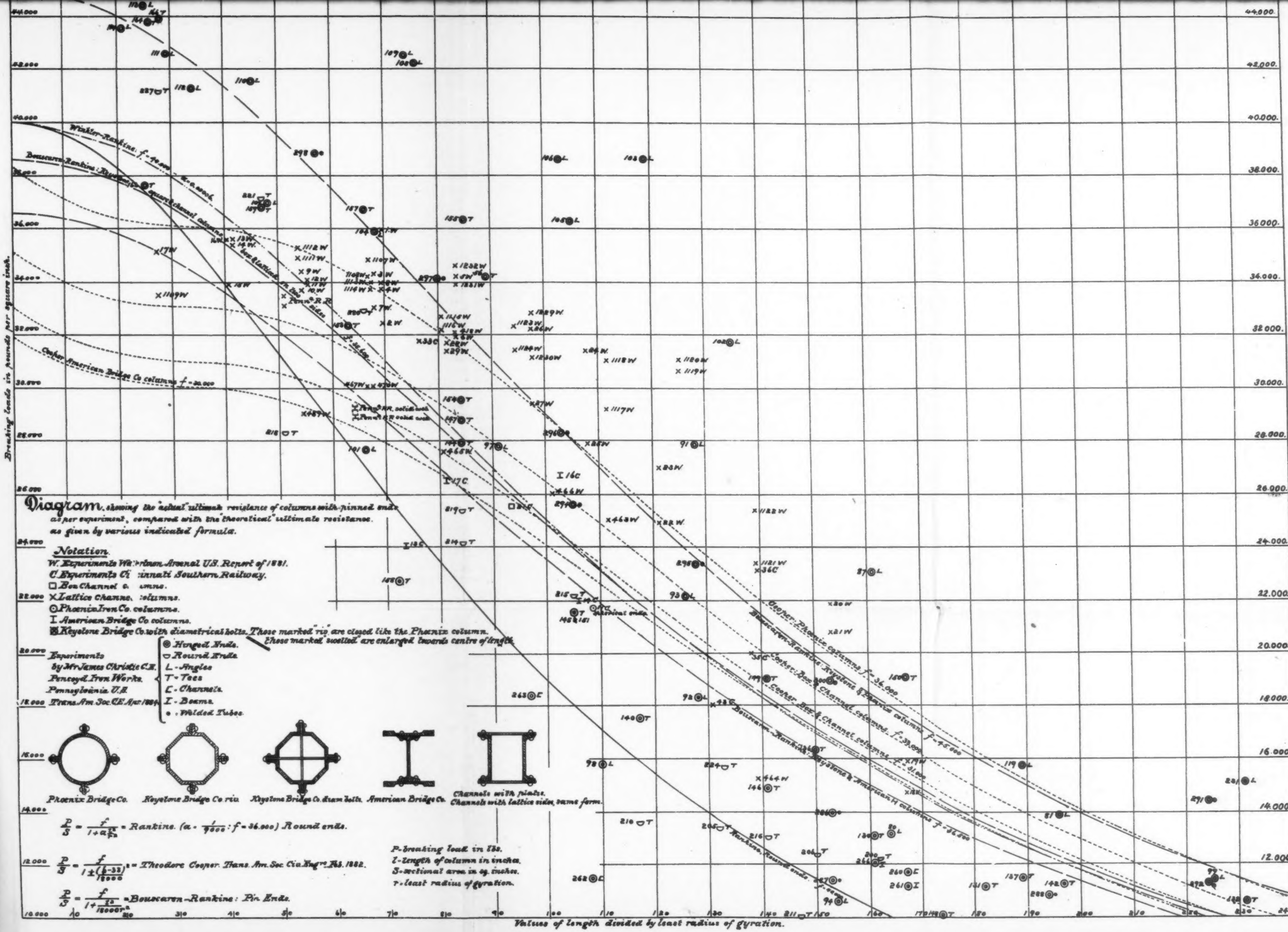
opt-


Values of length divided by least radius of gyration.
PLATE XXXV
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. Nº 335.
WILSON ON
STRENGTH OF IRON BRIDGES.
Breaking loads in pounds per square inch.
Breaking loads in pounds per square inch.
Bouscaren: Rankine: (where f = 38,000; a = ...)
Cooper: Phoenix columns f = 36,000.
Cooper: Box & Channel columns f = 31,000
Cooper: Box & Channel columns f = 33,000.
Bouscaren: Rankine: Keystone & American Bridge Co. columns
Winkler: Rankine
Diagram showing the "actual" ultimate resistance of columns with fixed ends, as per experiment, compared with the "theoretical" ultimate resistance, as given by various indicated formulae.
Notation.
P. Phoenix Iron Co. columns. Phoenixville U.S. tested at Watertown U.S. Trans. Am. Soc. Civ. Engrs: Feb. 1882.
W. Experiments Watertown Arsenal. U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway made in Chicago and Pittsburgh U.S. - G. Bouscaren C.E. Trans. Am. Soc. C.E. Dec. 1880.
□ Box Channel columns
× Lattice Channel columns.
⊙ Phoenix Iron Co. columns.


Diagram, showing the "actual" ultimate resistance of columns with fixed ends, as per experiment, compared with the "theoretical" ultimate resistance, as given by various indicated formulae.
Notation.
P. Phoenix Iron Co. columns. Phoenixville U.S. tested at Watertown U.S. Trans. Am. Soc. Civ. Engrs. Feb. 1882.
W. Experiments Watertown Arsenal. U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway made in Chicago and Pittsburgh U.S. - G. Bouscaren C.E. Trans. Am. Soc. C.E. Dec. 1880.
□ Box Channel columns
× Lattice Channel columns.
⊙ Phoenix Iron Co. columns.
I American Bridge Co. columns.
Keystone Bridge Co. with diametrical bolts. Those marked "riv." are closed like the Phoenix column: those marked "swelled" are enlarged towards centre of length.
Experiments by Mr. James Christie C.E. Pencoyd Iron Works. Pennsylvania U.S. Trans. Am. Soc. C.E. Apr. 1884.
△ Flat Ends.
Fixed Ends.
L - Angles.
T - Tees.
[- Channels.
I - Beams.
W.[- Watertown Channels.
• - Welded Tubes.
Phoenix Bridge Co.
Keystone Bridge Co. riv.
Keystone Bridge Co: diam. blts.
American Bridge Co.
Channels with plates. Channels with lattice sides same form.
$\frac{P}{S} = \frac{f}{1+a\frac{l^2}{r^2}}$ = Rankine. ($a = \frac{1}{36000}$: $f = 36000$)
$\frac{P}{S} = \frac{f}{1 \pm \frac{(\frac{l}{r}-80)^2}{18000}}$ = Theodore Cooper. Trans. Am. Soc. Civ. Engrs. Feb. 1882.
$\frac{P}{S} = \frac{f}{1+\frac{l^2}{36000 r^2}}$ = Bouscaren - Rankine.
P = breaking load in lbs.
l = length of column in inches.
S = sectional area in sq. inches.
r = least radius of gyration.
Breaking loads in pounds per square inch.
Values of length divided by least radius of gyration.
Bouscaren - Rankine: (when f = 38000; a = 1/...)
Cooper: Phoenix columns f = 36000.
Cooper: Box & Channel columns f = 33000.
Cooper: Box & Channel columns f = 31000.
Rankine: f = 40000.
Winkler Rankine: f = 40000; a = 0.00004.
Bouscaren - Rankine: Keystone & American H columns f = 36500.
Cooper: American Bridge Co. columns f = 30000.



Values of length divided by least radius of gyration.
PLATE XXXVI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. Nº 335.
WILSON ON
STRENGTH OF IRON BRIDGES.
Breaking loads in pounds per square inch.
Diagram, showing the "actual" ultimate resistance of columns with pinned ends as per experiment, compared with the "theoretical" ultimate resistance as given by various indicated formulæ.
Notation.
W. Experiments Watertown Arsenal U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway.
□ Box Channel columns.
× Lattice Channel columns.
⊙ Phœnix Iron Co. columns.
I American Bridge Co columns.
Keystone Bridge Co. with diametrical bolts. Those marked "r's" are closed like the Phœnix column.
those marked swelled are enlarged towards centre of length.
Experiments by Mr James Christie C.E. Pencoyd Iron Works, Pennsylvania U.S. Trans. Am. Soc. C.E. Apr. 1884.
⊙ Hinged Ends.
◡ Round Ends.
L - Angles
T - Tees
C - Channels
I - Beams
Winkler-Rankine
Bouscaren-Rankine
Cooper-American Bridge Co columns f = 30,000
Cooper-Phœnix columns f = 36,000



Diagram, showing the "actual" ultimate resistance of columns with pinned ends as per experiment, compared with the "theoretical" ultimate resistance as given by various indicated formula.
Notation.
W. Experiments Watertown Arsenal U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway.
□ Box Channel columns.
× Lattice Channel columns.
⊙ Phoenix Iron Co. columns.
I American Bridge Co columns.
⊠ Keystone Bridge Co. with diametrical bolts. Those marked "riv" are closed like the Phoenix column.
Those marked swelled are enlarged towards centre of length.
Experiments by Mr. James Christie C.E. Pencoyd Iron Works. Pennsylvania U.S. Trans. Am. Soc. C.E. Apr. 1884.
⊚ Hinged Ends.
▽ Round Ends.
L - Angles
T - Tees
C - Channels.
I - Beams.
• - Welded Tubes.
Phoenix Bridge Co.
Keystone Bridge Co riv.
Keystone Bridge Co. diam bolts.
American Bridge Co.
Channels with plates. Channels with lattice sides same form.
P/S = f / (1 + a l²/r²) = Rankine. (a = 1/9000 : f = 36.000) Round ends.
P/S = f / (1 ± ((l/r − 33)/9000)²) = Theodore Cooper. Trans. Am. Soc. Civ. Engrs. Feb. 1882.
P/S = f / (1 + l²/(18000 r²)) = Bouscaren-Rankine: Pin Ends.
P - breaking load in lbs.
l - length of column in inches.
S - sectional area in sq. inches.
r - least radius of gyration.
Winkler-Rankine.
Bouscaren-Rankine: Keystone & square channel columns
Cooper American Bridge Co columns f = 30.000
Cooper-Phoenix columns f = 36.000
Bouscaren-Rankine Keystone & Phoenix columns f = 45.000
Cooper: Box Channel columns f = 39.000
Rankine, Round ends.
Values of length divided by least radius of gyration.
Breaking loads in pounds per square inch.
44.000
42.000
40.000
38.000
36.000
34.000
32.000
30.000
28.000
26.000
24.000
22.000
20.000
18.000
16.000
14.000
12.000
10.000
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
210
220
230
240

ed the following formulas (Nos. 3, 4 and 5 of the specification) for the allowable working stress in compression per square inch of section.

For both ends fixed:

$$b = \frac{a}{1 + \frac{l^2}{36\,000r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For one end hinged:

$$b = \frac{a}{1 + \frac{l^2}{24\,000r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

For both ends hinged:

$$b = \frac{a}{1 + \frac{l^2}{18\,000r^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where a = permissible stress previously found.

b = allowable working stress per square inch.

l = length of piece in inches, center to center of connections.

r = least radius of gyration of the section in inches.

These formulas are identical with those designated here as Bouscaren-Rankine, except that they are adapted to finding directly the working compressive stress from a value previously determined, which may be termed the permissible stress per square inch for a column of one diameter, or more correctly perhaps, of a length equal zero.

Plate XXXVII shows diagrams of the working stresses for pin-ended columns as determined from the formulas of Cooper, Bouscaren-Rankine and Weyrauch, plotted with values of $\frac{l}{r}$ for abscissæ and the working stresses per square inch for ordinates. The same diagram shows the plotted curve of working stresses by the author's formula, equation (5), the value of a being taken from eq. (1) with $\frac{\text{Min.}}{\text{Max.}} = 0.31$ very nearly, b for $\frac{l}{r} = 0$ becoming 8 500 pounds. This curve will of course be found identical with that of Bouscaren-Rankine, when the ultimate stress for short prisms is taken at 42 500 pounds, and the factor of safety, as specified by Mr. Bouscaren, at 5.

Pieces used in compression, which are continuous over points of support, are generally free, so that they may deflect from the action of

the compressive force in one direction in one panel of the structure and in the other direction in adjacent panels, thus producing all the effects of hinging at the ends. The specifications therefore provide that such pieces, unless so firmly fixed as to be incapable of such bending, shall be considered as hinged at the ends.

The author places great importance upon having the lines of the neutral axes of all members which connect together meet at the same point; and, if there is a pin connection, the pin should also occupy as nearly as possible this same position. After providing for this, the specifications go on to say how the upper chords of deck bridges shall be proportioned when the floor system rests directly on them, and states that all other members subject to direct stress in addition to the bending moment shall be similarly calculated. In this connection it may be stated that the average maximum effect by the transverse load from an engine on the upper chord considered as a continuous beam, will be found to be about three-fourths what it would be if the beam was supported only, and the moment is generally from a single concentrated load, although this depends altogether upon the length of panel.

The specification now follows with the requirements for the sizes of eyes on all pin-connected tensile members, and these proportions are considered quite sufficient when the best known methods of manufacture are used. In some cases, however, larger heads might be found necessary, and this is covered by a clause under " Quality of Material," which specifies that all links and rods, if tested to the breaking, shall part through the body and not through the head or pin-hole. It is possible that the future may see very considerable reduction from the proportions as specified, when better methods of manufacture are devised. Indeed, the Edge Moor Iron Co., of Wilmington, Delaware, is now working eye-bars in both steel and iron by a new method of upsetting, without buckling or welding, which has given astonishing results. That company furnishes these bars with fifty per cent. excess in the width of head over bar when the diameter of the pin is less than the width of bar, and forty per cent. when the diameter of the pin exceeds the width of bar (really reversing the rule of the present specification when the diameter of the pin exceeds width of bar), and states that it will guarantee that the eyes will develop the full value of the bar even at only forty per cent. excess in head for the former case, and thirty-three per cent. for the latter. Experiments have been made at Edge

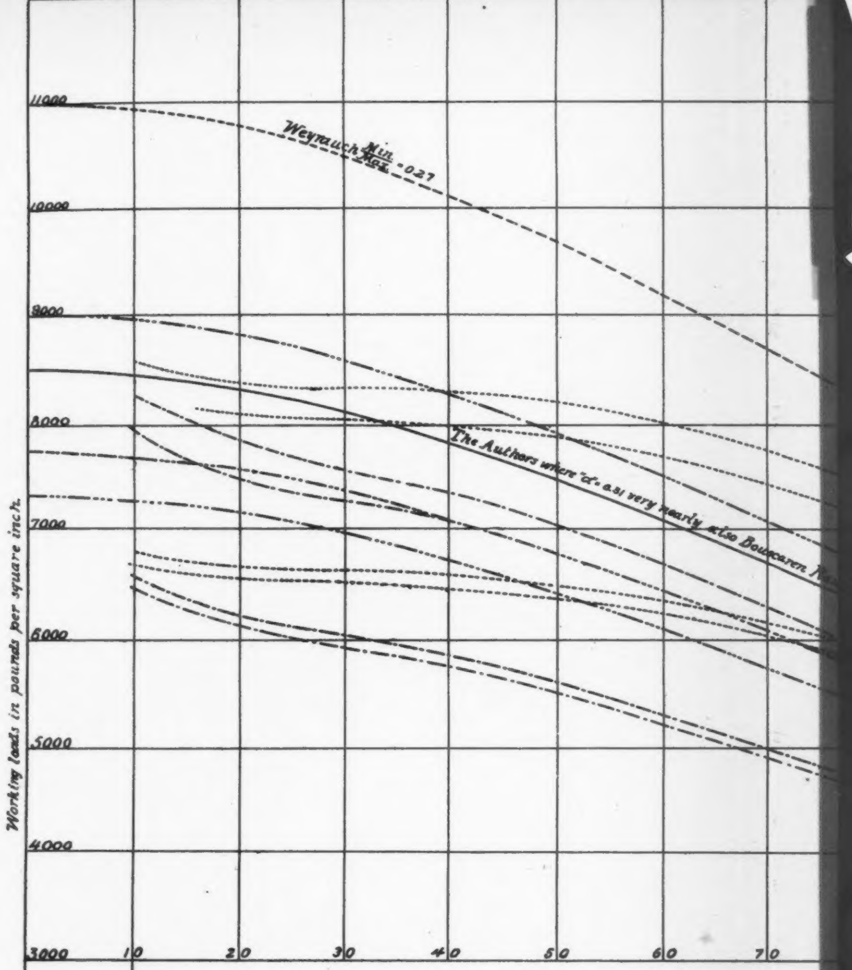


Diagram showing Working Stresses of Pin ended columns by various indi

a ———— Theodore Cooper. Trans. Am. Soc. Civ. Engrs. Feb. 1882. p. 85. $\frac{P}{S} = \frac{f_s}{1 \pm (\frac{l}{r} - 35}{18000}$

b —·—·— Theodore Cooper. Trans. Am. Soc. Civ. Engrs. Feb. 1882 p. 86 $\frac{P}{S} = \frac{f_s}{1 + (\frac{l}{r} - 35}{18000}$

$\frac{P}{S}$ = allowed working stress per square inch.

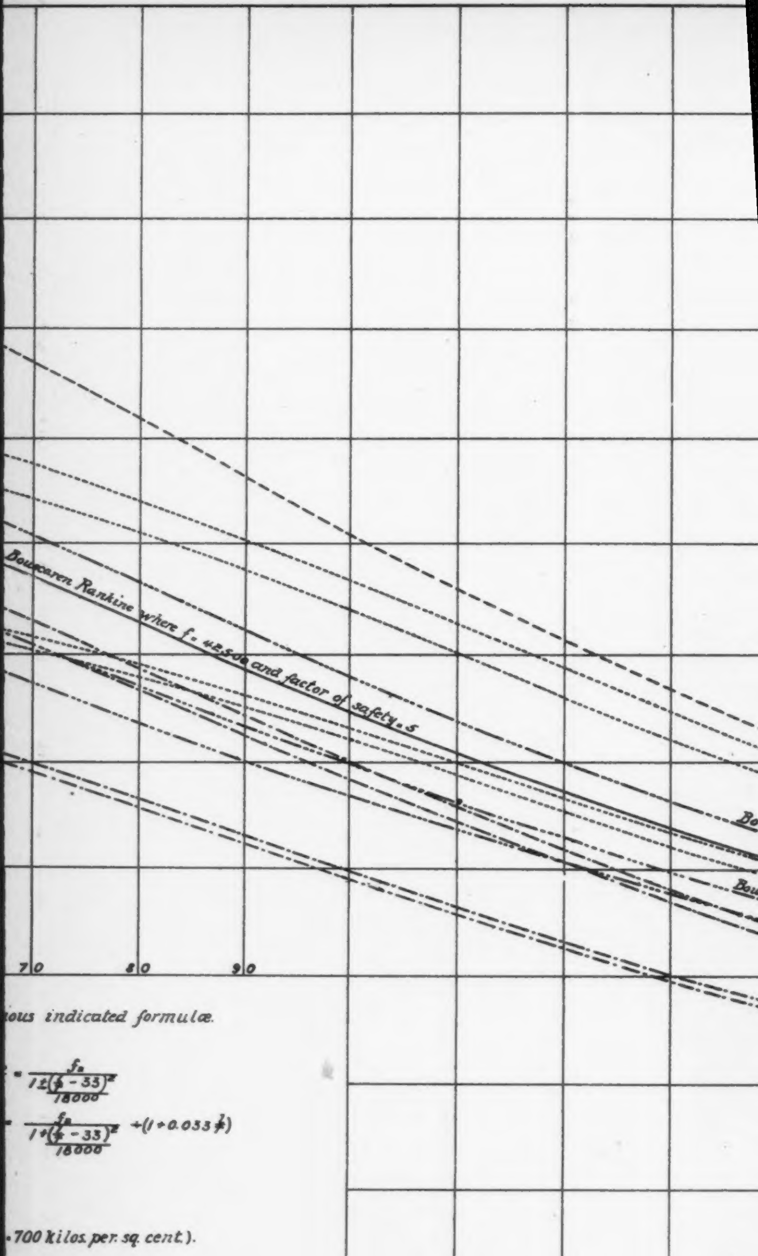
f_s = coefficient of working stress.

$\frac{l}{r}$ = ratio of length to least radius of gyration.

- - - - - Weyrauch (Proc. Inst. Civil. Engrs. Vol. LXIII. p. 292. eq. 38: v = 700 kilos

—— — —— Bouscaren, Rankine. factor of safety of five.

———— The Author and also Bouscaren, Rankine for special assumptions.



Boucaren Rankine where f = 42500 and factor of safety = 5
Bo
Bou
70
80
90
ious indicated formulæ.
$= \frac{f_0}{1 \pm \frac{(\frac{l}{r} - 33)^2}{18000}}$
$= \frac{f_0}{1 + \frac{(\frac{l}{r} - 33)^2}{18000}} \div (1 + 0.033 \frac{l}{r})$
= 700 kilos per sq. cent.).
mptions.
70
80
90
100
110
120
130
Values of length divided by least radius of gyration.

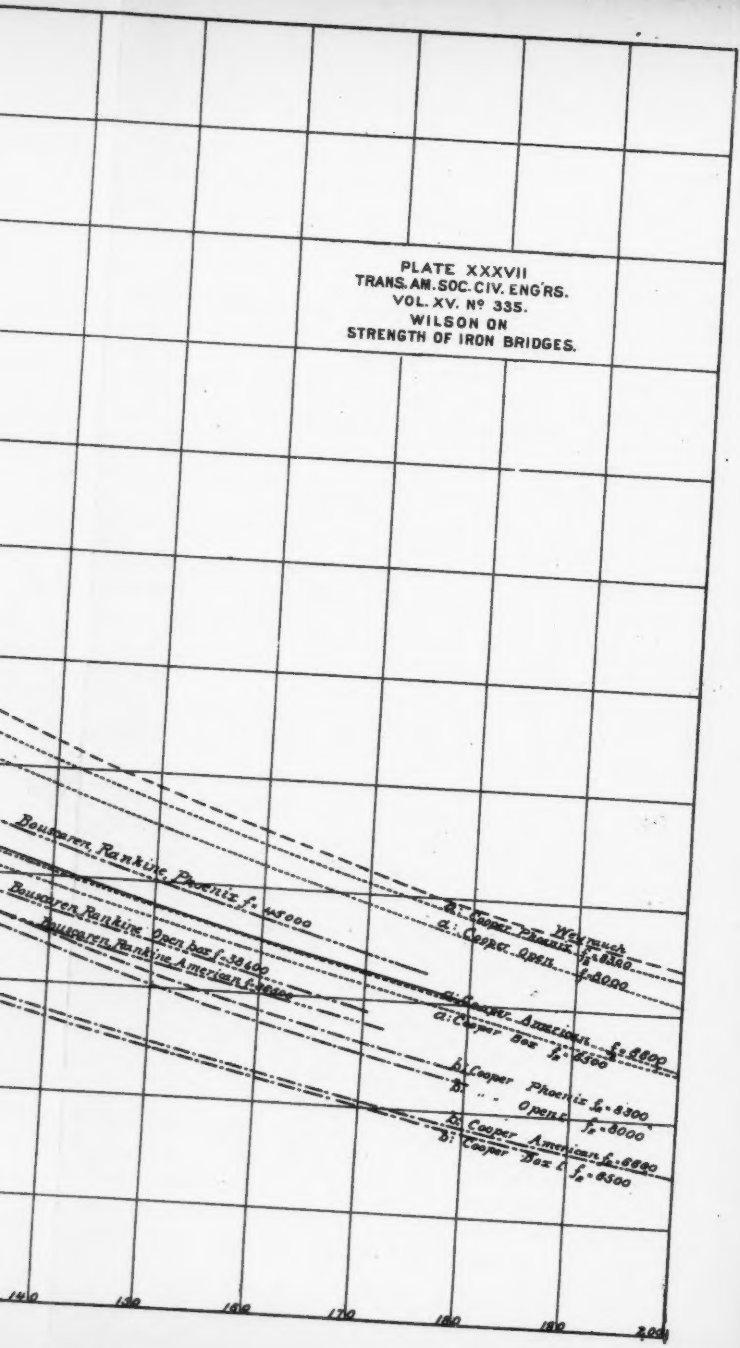


PLATE XXXVII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. Nº 335.
WILSON ON
STRENGTH OF IRON BRIDGES.
Bouscaren, Rankine, Phoenix f = 45000
Bouscaren Rankine Open box f = 38600
Bouscaren Rankine American f = 36500
a: Cooper Weyrauch
a: Cooper Phoenix f = 8300
a: Cooper Open f = 8000
a: Cooper American f = 6800
a: Cooper Box f = 6500
b: Cooper Phoenix f = 8300
b: " " Open f = 8000
b: Cooper American f = 6800
b: Cooper Box f = 6500
140
150
160
170
180
190
200

Moor on bars of this mode of manufacture having an excess in width of head as low as 20 per cent., and the bar has still parted through the body under the ultimate stress.

The table on page 400 is a record of some tests which have been recently made. The minimum limit has evidently not yet been reached.

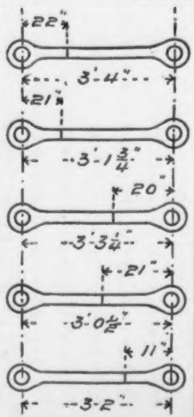
The rule given for the minimum diametrical bearing between pins and pin-holes is an old one, and experience has proved it satisfactory. It is preferred not having pins less in diameter than three-fourths the width of the bar.

The author considers it very essential that the pins be so proportioned and the pieces connected by them so packed, that the maximum stress per square inch on the outer fibers of the pins from the cumulative bending moments shall not exceed one and a half times the maximum tensile stress *a* in the members connected. It is true that many old bridges are yet in service where the pins would not by any means come up to this requirement of the specification, but there is no doubt that the stresses in the links are very unevenly distributed, and some of the pieces may be seriously overstrained. In such bridges it is very difficult to determine whether the results are dangerous, but they must certainly produce an effect of deterioration on the structure which may end seriously.

Among the tests already referred to as made for the author at Watertown Arsenal, was one on pin-connected links, to ascertain the effect of the bending of the pin on the links which it connected. It is hardly necessary at the present time to give a detailed account of this test, but it may be said to have fully confirmed the views already expressed.

In arranging the spacing of rivets, it will be noticed that the various details, etc., are to be constructed so that the compression members in these detailed parts are good for a working stress as high as would be obtained in a column of twelve diameters. In limiting the distance of rivets apart at right angles to the line of stress, at not more than thirty times the thickness of the thinnest external plate, it is desired to prevent any liability to buckling from stresses, which would otherwise be perfectly legitimate on the members. Stoney ("Theory of Strains," London, 1869, p. 210), in referring to the results of Mr. Hodgkinson's experiments on compression of wrought-iron in rectangular tubes, where the yielding is by buckling, not flexure, leads to the conclusion that the

REPORT OF TESTS ON STEEL EYE-BARS MANUFACTURED BY EDGE MOOR IRON CO., FROM BESSEMER STEEL MADE IN PENNSYLVANIA.



Date of Test, 1885.	Mark.	Length of Bar and Location of Break.		Percentage of Excess in Heads.		Size of Bar.	Area in Square Inches.	Diameter of Eye.		Diameter of Pin.		Limit of Elasticity in pounds per square inch.	Percentage of Elongation in length of 8 inches.	Percentage of Reduction of Area.	Ultimate Strength in pounds per square inch.	Elongation of Pin-hole.		Remarks as to Fracture, etc.
		Eye A.	Eye B.	A.	B.			A.	B.	A.	B.					A.	B.	
April 22..	No. 2			24.1	36.6	5" x1 1/16"	5.15	12"	12"	5 15/16"	5 7/16"	38 800	34.3	52.3	67 600	0.5"	0.14"	Silvery cupped with some granular spots.
" 23..	" 10			21.4	20.2	4" x1"	4.	10"	10"	5 3/16"	5 3/16"	37 000	37.5	58.2	61 100	1.51"	0.81"	Silvery cupped.
" "..	" 11			26.5	26.5	"	4.	10"	10"	4 15/16"	4 15/16"	38 900	31.2	59.8	63 000	0.82"	1.36"	" "
" "..	" 13			26.1	26.	5" x1"	4.85	11 1/2"	11 1/2"	5 7/16"	5 7/16"	41 200	40.6	55.1	61 100	0.79"	1.88"	" "
" "..	" 15			23.5	23.3	4" x1"	4.	10"	9 3/8"	5 1/16"	4 7/16"	38 900	31.2	44.2	60 400	1.41"	0.94"	2/3 Silky, 1/3 Granular

thickness of plate for proper resistance should be not less than one-thirtieth of the breadth of the tube, or in the present application, of the distance between the lines of rivets by which the parts are assembled together. Where straps and plates are used merely for stiffening, and do not take part in the compressive resistance of the column, their thickness is limited to not less than one-fortieth, and for double straps, one-fiftieth of the distance between the rivets connecting them to the compressed members. There is very little positive data on which to base these rules for widths of plates, beyond practical experience and professional opinion. It is hoped that further experiments with built compression members will develop more reliable information. It has been claimed by some designers that by omitting the area of the connecting plate in arranging the required section of a column, this plate may be made thinner than if it was included. In reply to this it may be said that so long as this plate is continuous, it is not possible to omit it; that its apparent omission only increases the actual section of the column by that much, and thereby lessens the amount of stress a certain small proportion over the whole of it, the connecting plate included, but the latter still takes its proportion of strain.

As to the proper distance for rivets from the sides and ends of pieces, the specification, while cautionary, does not give any definite rule, but the author himself follows the rules of Edge Moor Iron Company. Detailed rules are given for lattice strapping and terminal cross-bracing plates, as determined by custom and experience.

The shear on the net section of any member is limited to the compressive working stress a, and for rivets twenty per cent. extra section must be allowed to cover inaccuracy in workmanship, etc. This agrees with general practice. It is required that no allowance shall be made for the web in calculating the flange sections of plate girders. The author is aware that the web is often included, and in the case of solid rolled beams the specification allows it. In built girders, however, which generally have thin webs, the author considers such practice as working very closely, the gain from it being very small and the material so used being really put in the structure for another purpose.

The succeeding clauses of the specification concerning transverse stress, also the placing of rivets, etc., in plate girders, require little or no explanation. In reference to the proper method of proportioning the web and stiffeners of plate girders there seems to be a difference of

opinion among authorities, or at least a want of clearness in explanation, and a failure to give the subject its most economical consideration, at least in theory. Professor Rankine gives a rule for proportioning the web, in which he considers the shearing stress at the neutral axis as equivalent to a pull and a thrust of equal intensity inclined in opposite ways at 45 degrees from the vertical, the web tending to give way by buckling under the thrust. The vertical shear being resolved into these two components, one of tension and the other of compression on the web, the latter is computed to resist the compressive force as a fixed column of rectangular section, with a length equal to the distance along a line inclined at 45 degrees between two of its vertical stiffening ribs; or, if it has no such ribs, between the upper and lower horizontal flanges, a factor of safety of six being employed. When ribs are introduced, Professor Rankine is hardly as clear as he might be. He considers these either as suspending pieces (ties) or pillars (struts), according to the position of the load either on the lower or upper flange, and appears to proportion them for the local loading, those at the ends of the girder being the only ones specified to take the entire proportions of the load which rests at those points, or in other words, the vertical shear. The ribs act by their stiffness to prevent buckling of the web, and in that sense, if placed sufficiently close together, by reducing the length of column of the web, lessen its required thickness. Otherwise the thickness of web is computed in the same way, with or without ribs. Mr. Stephenson seems to have had very much the same opinion as expressed by Professor Rankine, and Professor Airy in his investigations on the subject also had the same views, that the girder acted as a lattice truss of an infinite number of intersections, these making up the web, the material of which could sustain, without injury, forces of tension and compression acting at right angles to each other, the least resistance being to the compression.

So long as no stiffeners are used, this rule of Professor Rankine's appears rational and practical, giving satisfactory results. When, however, the girder becomes too deep for an economical and practical thickness of web, stiffeners are required, and on the above theory they must be close enough together to limit the ratio of length of column to thickness to an economical figure. A girder of 60 feet length and 5 feet depth of web for example, sustaining a load of 3 000 pounds per foot lineal, would require the stiffeners about 14 inches apart for a three-eighths web.

The author in his early experience, twenty-five years ago, was very much troubled by such results when he knew of girders of the above dimensions and loading, in actual use, having webs of only $\frac{3}{16}$ of an inch in thickness and stiffeners at intervals of 5 feet, these girders bearing up well under use. Some of them, now thirty years old, are still in service, although it is not claimed that they would come within the present specification. The author soon discovered, however, by means of a paper model with very thin flexible web, that when stiffeners were properly introduced the web no longer resisted by compression, but by tension, the stiffeners taking up the duty of compressive resistance, like the posts of a Pratt truss, and dividing the girder into panels equivalent to those of an open truss, the web in each panel acting as an inclined tie. Working on this theory, results were obtained that quite agreed with practical examples.

Stoney, in his "Theory of Strains" (London, 1869, p. 319), approaches nearer to the true elucidation of the problem, and appears to understand the matter, although he is hardly definite and positive enough as to the course that should be pursued.

Mr. Theodore Cooper, M. Am. Soc. C. E., in an article on the rules for pitch of rivets and thickness of web in riveted plate girders (*Van Nostrand's Engineering Magazine*, Vol. XVII, p. 209), gives a rule very similar in its operations to that of Prof. Rankine, but in which he uses a factor of safety of only one-fourth, and for the extreme limit allows a length of column corresponding to a working resistance of 3 000 pounds per square inch.

While Mr. Cooper appears to introduce his vertical stiffeners properly for the whole shearing force, he would require for the web a much greater thickness, or would, it seems to the author, be obliged to introduce stiffeners considerably closer than practical experience would justify. Thus, for the example previously quoted, of a girder 60 feet span and 60 inches deep, the thickness of web required to resist buckling would be, on his rule, $\frac{3}{4}$ of an inch, or to use a $\frac{3}{8}$-inch web, the author presumes Mr. Cooper would place stiffeners $2\frac{1}{2}$ feet apart. Believing, as the author does, that the web resists by tension and not by compression, he would be perfectly satisfied that this would be safe enough, but if he was obliged to consider the effect as compressive, he would not. If the limit of depth of web is taken at eighty times the thickness, the limit of the diagonal in which the force, if compressive,

would really act, and which is the length of the column of resistance, would be about 113. It is the fact of the web actually resisting by tension which saves the case.

The theory of the author is that the flanges, with a portion of the web, act to carry the load and to concentrate it at the vertical stiffeners, if it is not already there. The latter act as the vertical columns in a Pratt truss would do, carrying the shear, which is resolved on to the web in tension in a diagonal direction, panel by panel, towards the abutment. The conditions of loading will control the frequency of struts, as there is, of course, a limit to the ability of the flange to carry intermediate loads to the struts, and this should receive due consideration.

The requirements in reference to number, size, spacing, bearing, etc., of rivets, exercise so important a control over the thickness of web, that after it is determined for these, it will probably be found quite sufficient for anything that will otherwise come upon it, the stiffeners, of course, being properly placed.

The requirements of the specification for wind strains and the proper proportioning of the lateral system are, it is believed, in accordance with the latest American practice in this respect.

Great care has been taken in framing the conditions for Quality of Material and Workmanship to insure having what is desired and what the present advanced state of manufacture can produce, without asking for what it is impossible to obtain except at an extraordinary cost. A practical experience of some years and extended intercourse with those who are foremost in the manufacturing interests of our country have, of course, formed the basis for this portion of the specification.

APPENDIX.

Specifications for Strength of Iron Bridges.

The structure to be wholly of wrought-iron, and designed to carry the following live loads on each track, headed in the same direction on adjacent tracks.

First.—Two typical consolidation engines coupled, weighing, with tender, 86 tons each, distributed as shown in figure 1, page 406, connected with a train weighing 3 000 pounds per foot lineal of track; or

Second.—Two typical passenger engines, coupled, weighing, with tender, 88 tons each, distributed as shown in figure 2, page 406, and connected with a train weighing 3 000 pounds per foot lineal of track; or

Third.—One class M engine, weighing, with tender, 66½ tons, distributed as shown in figure 3, page 406, and connected with a train of 3 000 pounds per foot lineal of track. The maximum given by either of these methods of loading is to be used in proportioning every member of the structure.

In calculating web-members of trusses and girders, the cross-girder load under the drivers is to be considered as the head of the train, the load on the preceding cross-girder being neglected.

In addition to the live loads before mentioned, the structure shall carry the following dead load, viz.:

At the panel points of the loaded chords:

First.—The weight of the floor (composed of a, the weight of the cross-ties used for the particular kind of floor adopted, and b, a weight of 140 pounds per foot lineal of track—covering the weight of rails, guard-rails, splices, spikes and bolts).

Second.—One-half the weight of the truss.

Third.—The weight of the iron floor system, if any.

Fourth.—The weight of the lateral system belonging to the loaded chord; and

Fifth.—One-half the weight of the sway bracing.

At the panel points of the unloaded chords:

First.—One-half the weight of the truss.

Second.—The weight of the lateral system belonging to the unloaded chord; and

Third.—One-half the weight of the sway bracing.

The span for calculation is to be taken from center to center of end pins, or from center to center of abutment plates or other supports; and the height from center to center of chord pins in truss bridges, or between centers of gravity of flange sections in plate girders, provided it does not exceed the distance out to out of angles, in which case the latter amount shall be considered the height.

The maximum and minimum stresses in compression and tension, as found for the before-mentioned loads, are to be used in determining the permissible working stress in each piece of the structure, according to the following formulas:

For pieces subject to one kind of stress only (all compression or all tension):—

$$a = u\left(1 + \frac{\text{minimum stress in member.}}{\text{maximum stress in member.}}\right) \qquad (1)$$

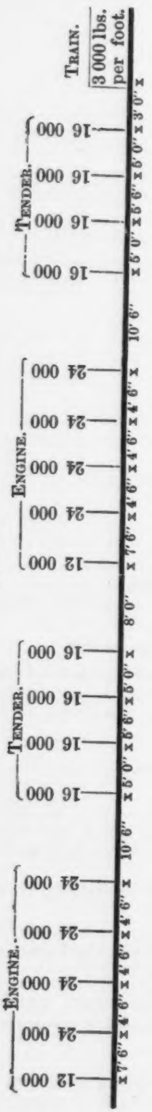


Fig. 1.

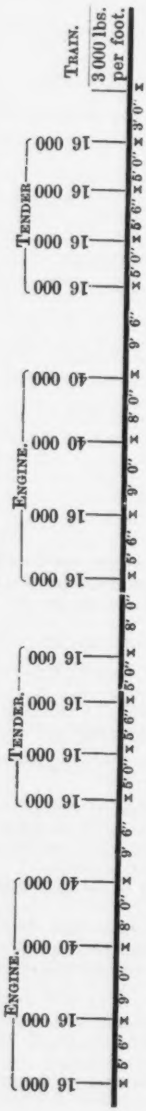


Fig. 2.

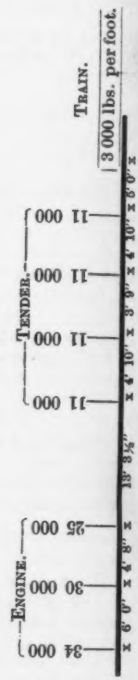


Fig. 3.

For pieces subject to stresses acting in opposite directions :--

$$a = u\left(1 - \frac{\text{maximum stress of lesser kind.}}{\text{2 maximum stress of greater kind.}}\right) \qquad (2)$$

In the above formulas :—

a = permissible stress per square inch, either tension or compression.

u = for double-rolled iron in tension (links or rods) 7 500 pounds per square inch.

u = for rolled iron in tension (plates or shapes) 7 000 pounds per square inch.

u = for rolled iron in compression 6 500 pounds per square inch.

The permissible stress a for members in compression is to be reduced, in proportion to the ratio of the length to the least radius of gyration of the section, by the following formulas :—

For both ends fixed $$b = \frac{a}{1 + \frac{l^2}{36\,000\, r^2}} \qquad (3)$$

For one end hinged $$b = \frac{a}{1 + \frac{l^2}{24\,000\, r^2}} \qquad (4)$$

For both ends hinged $$b = \frac{a}{1 + \frac{l^2}{18\,000\, r^2}} \qquad (5)$$

Where a = permissible stress previously found.
b = allowable working stress per square inch.
l = length of piece in inches center to center of connections.
r = least radius of gyration of the section in inches.

Pieces used in compression which are continuous over points of support are to be considered as hinged at the ends, unless so firmly fixed in direction as to be incapable of bending in opposite directions on the opposite sides of the points of support :—

In all cases where possible, the lines of the neutral axes of all pieces meeting at a joint must be made to meet in the same point, and where pins are used to form connections, they must be placed as nearly as possible in the neutral axes of the sections.

When not so arranged, provision must be made for taking up the bending stresses produced.

When the floor system rests directly on the upper chords of deck bridges, the said chords shall be so proportioned that the algebraic sum of the stresses per square inch on the outer fibers [due,

First.—To the weight of that part of the floor system which is supported by the chord (considered as acting on a continuous beam of a span equal to the panel length).

Second.—To the direct thrust.

Third.—To three-fourths of the maximum bending produced by that portion of an engine of the heaviest class which is supported

by the chord on a span equal to the panel length (considered as a supported beam); and

Fourth. (In case the pin is not in the neutral axis of the chord.) —To the algebraic sum of the moments of all chord stress increments acting at centers of pins.]

shall not exceed at the panel point the working stress a, or shall not exceed at the center of the panel the working stress b.

All other members which are subject to direct stress in addition to bending moment are to be similarly calculated.

Built chords must be thoroughly spliced and the splices riveted in the field, not bolted.

The eyes on all tensile members shall have fifty per cent. excess of material at the pin when the diameter of the pin does not exceed the width of the bar, and one hundred per cent. excess when the diameter is twice the width of the bar or over. For intermediate sizes of pins the excess of eye may be made proportional to their diameter.

The diametrical bearing between pins and pin-holes (diameter of pin $\times$ thickness of bearing) shall not be less in area than

$$\frac{2}{3}\left(\frac{\text{maximum stress in member.}}{\text{compressive unit stress } a \text{ for that member.}}\right)$$

Eye plates must have a sufficient size and number of rivets to properly distribute the bearing stress from the pins to members of the truss.

Pins are to be so proportioned that the maximum stress per square inch on the outer fibers (calculated from the cumulative moments of the stresses acting on the pieces connected, and the moment of resistance of the pin directly) shall not exceed one and one-half times the maximum tensile stress a in the members connected.

All rods with screw ends must be upset, and if ordinary nuts are used they must be double. Rods provided with adjusting screws are each to have an amount of five tons added to the calculated stress to allow for initial stress.

Floor-beam hangers must have an additional section of twenty-five per cent. above that given by the before mentioned limiting stresses.

Rivets must be so spaced that they shall not be further apart in the direction of the stress than twelve times the thickness of the thinnest external plate connected, and not more than thirty times that thickness at right angles to the line of stress.

Rivets must be kept a sufficient distance from the sides and ends of pieces to avoid any danger of splitting out, and not placed closer than three diameters center to center.

Single lattice straps shall have a thickness of not less than one-fortieth ($\frac{1}{40}$), and double straps connected by a rivet at the intersection, not less than one-fiftieth ($\frac{1}{50}$) of the distance between the rivets connecting them to the compressed members; and their width shall be :

For 15-inch and 12-inch channels, or equivalent built section ($\frac{7}{8}$-inch rivets), $2\frac{1}{2}$ inches.

For 15-inch and 12-inch channels, or equivalent built section ($\frac{3}{4}$-inch rivets), $2\frac{1}{4}$ inches.

For 10-inch and 9-inch channels, or equivalent built section ($\frac{3}{4}$-inch rivets), $2\frac{1}{4}$ inches.

For 8-inch and 6-inch channels, or equivalent built section ($\frac{5}{8}$-inch rivets), 2 inches.

For 8-inch and 6-inch channels (extra light sections) and 5-inch channels ($\frac{1}{2}$-inch rivets), $1\frac{3}{4}$ inches.

The distance between connections of the strapping shall be such that the individual members composing the column considered with hinged ends and a length equal to the distance between these connections shall be stronger than the column as a whole; and in no case shall this distance exceed eight (8) times the least width of these members.

All segments of members in compression connected by strapping only, shall have terminal cross-bracing plates at each end, the rivets and net section of which shall be sufficient to transfer the total maximum stress borne by the segment, and the thickness of which shall not be less than one-fortieth ($\frac{1}{40}$) of the distance between the rivets connecting them to the compressed members.

The shear on net section of any member shall not exceed the compressive working stress a, and in case of rivets at least twenty per cent. extra section must be allowed.

No allowance shall be made for the web in calculating the flange sections of plate girders.

The stresses in solid rolled beams shall be calculated from the moment of inertia of the section.

Flanges of plate girders running over twelve inches in width shall have at least four lines of rivets.

The stress in the outer fibers of I beams, channels, etc., subject to bending moments, shall not exceed the tensile working stress a for rolled shape iron.

In all cases for compressed flanges of beams or girders (subject to transverse stress), the permissible working stress in such flanges shall be computed by Rankine's formula:

$$c = \frac{a}{1 + \frac{l^2}{5000\ w^2}} \tag{6}$$

Where a = permissible stress previously found.
c = allowable working stress per square inch.
l = unsupported length in inches.
w = width in inches.

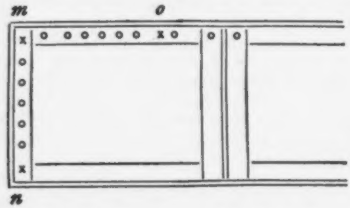
In no case shall a stress greater than that for a length equal to twelve times the width be used.

The rivets in plate girders shall be proportioned for shear as previously specified, and the rivets through the web and flange angles, and through the web and vertical stiffeners, at a splice, concentrated load or end of girder, must not have a bearing pressure per square inch against the web plate (on a diametrical section of the rivets) of more than twice the compressive stress a used in the upper flange of the girder; provided that they are not placed closer than twice their diameter from the edge of the web-plate to edge of hole. If placed closer than this the bearing pressure must be proportionally reduced.

In calculating the shearing stress and bearing stress on web rivets of plate girders, the whole of the shear acting on the side of the panel next the abutment is to be considered as being transferred into the flange angles in a distance equal to the depth of the girder.

The number of rivets in the flanges in the distance $m - o$ (equal $m - n$) shall be sufficient to transfer the shear at $m - n$ into the flange angles without exceeding the specified shearing stress or the bearing

stress on diametrical section of rivets, and the number of rivets in the distance $m - n$ shall follow the same rule.



Net sections must be used in all cases in calculating tension members, and in deducting rivet holes they must be taken one-eighth inch larger than the size of rivets.

In calculating the net section of angles in plate girders *all* the rows of rivets must be deducted, and in flange plates having rivets staggered, all rows must be deducted unless so arranged that the net section along a zigzag line, taking all distances in the diagonal direction at only three-fourths their value, exceeds the corresponding net section directly across the plate.

When the thickness of the web plate is less than one-thirtieth of the unsupported distance between flange angles, heavy stiffeners shall be riveted on both sides of the web with a close bearing against the upper and lower flanges, and calculated as columns for the whole shear at the several points where they are placed. These stiffeners, in girders over three feet in depth, shall be placed at distances apart (center to center) generally not exceeding the depth of the full web-plate, with a maximum limit of five feet. In girders under three feet in depth they may be three feet apart, and in some special cases, where there is little or no shearing stress, at a greater distance.

In every case at least one upper flange plate on plate girders shall extend from end to end of the same to give lateral stiffness, and any additional plates used to make up the flange section shall be made of such length as to allow of at least two rows of rivets, of the regular pitch, being placed at each end of the plate beyond the theoretical point required.

Girders formed with web-plate and angles alone, having no upper flange plate proper, will not be allowed.

All flange plates, subject to either tension or compression, spliced in the length of girder, must be covered by an amount of extra material equal in section to the pieces spliced, with sufficient rivets on either side to transmit the stresses from the parts cut.

Flange angles must be spliced with angle-covers whenever cut within the length of the girder, or else the amount of material cut must be replaced by an equal amount of extra material in the flange plate.

No iron shall be used of less thickness than one-quarter inch, no piece used in compression shall have an unsupported width of more than thirty times its thickness, and no plate girder web shall be less than three-eighths of an inch in thickness.

Continuous girders will not be permitted, except in the case of upper chords carrying floor and in drawbridges.

Through bridges must have a clear head room of not less than eighteen feet six inches from base of rail.

Standard inside clear width for full through bridges, single track on a tangent, is thirteen feet. Width for bridges on curves and half through bridges where girders project up above the track, is subject to modification, depending upon the special case.

Truss bridges are to be cambered with a rise of not less than $\frac{1}{1200}$ of their length; the cross ties to be sized down over the track girders, so that the camber line of track may be a true circular arc.

Cross ties to be of white oak, having a width of ten inches and a minimum depth of seven inches, and spaced not over twenty inches between centers, with every fourth tie bolted down by three-quarter inch bolts, having round flat heads and two hexagon nuts each. When track is curved, the outer rail to be elevated as may be required.

In the case of deck bridges with wooden floor-beams, when the distance between centers of supports exceeds six feet, the floor-beams (ties) are to be proportionally increased.

Guard rails of white oak or long-leaved southern pine, six by eight inches, are to be placed ten inches in the clear outside of each track rail; to be notched one and one-half inches over the cross ties, secured to every fourth tie by a three-quarter inch bolt, having flat round head, two hexagon nuts and flat wrought washer, and to all other ties by three-quarter inch square wrought spikes. Splices to guard rails to be twelve inches long, placed between ties, with the joint horizontal, two three-quarter inch bolts with flat round heads, wrought washers and double hexagon nuts being used for each.

Lateral bracing shall be proportioned for a wind pressure, acting in either direction horizontally, of thirty pounds per square foot on the whole surface of all trusses and the floor, as seen in elevation, in addition to a train of ten feet average height, beginning two feet six inches above base of rail, moving across the bridge; except in the case of through bridges, where the surface of the truss covered by the train may be deducted.

Where the bridge is on a curve, the lateral bracing, in addition to wind stress, must be proportioned to resist a centrifugal force due to as many trains as there are tracks, moving at the rate of sixty feet per second.

The whole of the wind stress due to the train and floor, plus one-half the truss, is to be considered as acting on the lateral bracing of the loaded chord, and that due to one-half the truss only on the lateral bracing of the unloaded chord. The end portal bracing in through bridges must be of sufficient strength to transfer the accumulated wind stress from the upper lateral system to the end posts, and the end sway bracing in deck bridges shall carry the whole of the accumulated wind and centrifugal stress from the loaded chord to the abutment, intermediate sway bracing being placed in each main panel and adapted to carry half the maximum stress-increment due to the wind on the train and to centrifugal force. In case of very heavy curves, some of the centrifugal force may be transferred to the lower lateral bracing.

In through plate-girders the portion of the train covered by the girder may be deducted from the wind surface, and only one girder and one train surface considered.

In all cases where the rods have adjustment, an addition to the above stresses of five tons must be made for initial tension.

Lateral rods in tension shall not be strained more than 15 000 pounds per square inch, and plate or shape iron not more than 12 000 pounds per square inch under the above conditions.

Lateral struts in compression shall not be strained more than 12 000

pounds per square inch (including the proper component of the initial tension allowed on the lateral rods at their extremities), and reduced in proportion to their length and least radius of gyration, as previously specified.

In case the maximum stresses in the chords of the bridge or flanges of floor-girders due to wind and centrifugal force (the chords and lateral bracing being considered as a truss lying on its side), added to the maximum stresses in the chords or flanges due to vertical loading, shall exceed the before mentioned limits of 15 000 and 12 000 pounds per square inch in tension and 12 000 pounds per square inch in compression, properly reduced, additions must be made to said chords or flanges until such limits are not exceeded.

Should the stresses in said chords be reversed in any possible case, proper provision must be made for such stress in an opposite direction.

No deduction shall be made from chord sections on account of material in lateral system, but the chords shall be made of the full section previously specified.

In every case the connections between the wind bracing and chords must be made of greater strength than the wind bracing itself, and so designed as to avoid as far as possible inducing bending moments in any members of the structure, and such connections must be capable of transferring the longitudinal components of the wind stresses into the main truss chords in a direct and satisfactory manner, or a separate chord be used for the lateral system.

The trusses must be secured from side motion on bearing plates, and must have ample bearing and roller support, the weight on the rollers not to exceed 750 $\sqrt{d}$ pounds per lineal inch, d being the diameter in inches. Girder bridges less than sixty-five feet opening will not require rollers. The bolster blocks must be joined to the truss. The bearing plates must be secured to the underlying support by bolts or dowels. Bearing plates shall not give a greater pressure on masonry than 300 pounds to the square inch, unless in specially authorized cases.

In the case of trestles or iron piers, they shall be proportioned for vertical load under the same limiting stresses given for trusses, and for wind stresses and centrifugal stresses, loading and bending combined, the stresses shall not exceed those given for lateral bracing.

In addition to the above, the structure shall be capable of resisting wind pressure on its exposed surface alone of fifty pounds per square foot without exceeding the limiting stresses for lateral bracing.

Tension at the foot of the windward column is to be avoided if possible, and in any case approved anchor bolts well secured in the masonry shall be used.

All strain sheets and plans must be submitted to the engineer of the company in charge of bridges for approval, and a complete set, including details, furnished to him without charge. All details to be subject to his approval, and access to be allowed him or his assistants to the contractors' working drawings and shop for examination of details.

Quality of Material.—All wrought-iron must be tough, fibrous, uniform in quality throughout, free from flaws, blisters and injurious cracks, and must have a workmanlike finish. It must be capable of sustaining an ultimate stress of 46 000 pounds per square inch on a full section of test-piece, with an elastic limit of 23 000 pounds per square inch.

All iron to be used in tension or subjected to transverse stress (except web-plates), must have a minimum stretch on a length of eight inches of fifteen per cent. measured after breaking.

All iron to be used in compression and for web-plates, of width not exceeding twenty-four inches, must have a minimum stretch of ten per cent. on a length of eight inches measured after breaking.

All iron for web-plates exceeding twenty-four inches in width, must havea minimum stretch of five per cent. measured in length of eight inches.

All iron to be used in the tensile members of open trusses, laterals, pins, bolts, etc., must be double-rolled after and directly from the muck bar (no scrap will be allowed) and must be capable of sustaining an ultimate stress of 50 000 pounds per square inch on a full section of test-piece, with an elastic limit of 25 000 pounds per square inch and a minimum stretch of twenty per cent. measured after breaking in length of eight inches.

When tested to the breaking, if so required by the engineer, the links and rods must part through the body and not through the head or pin-hole. Such tests must be at the expense of the contractor when the requirements of these specifications are not complied with.

All tension wrought-iron, if cut into testing strips one and one-half inches in width, must be capable of resisting, without signs of fracture, bending cold by blows of a hammer, until the end of the strips form a right angle with each other, the inner diameter of the curve of bending being not more than twice the thickness of the piece tested. The hammering must be only on the extremities of the specimens, and never where the flexion is taking place. The bending must stop when the first crack appears.

All the tension tests are to be made on a standard test piece of one and one-half inches in width, and from one-fourth to three-fourths inches in thickness, planed down on both edges equally, so as to reduce the width to one inch for length of eight inches. Whenever practicable, the two flat sides of the piece to be left as they come from the rolls. In all other cases both sides of the test pieces are to be planed off.

All plates, angles, etc., which are to be bent in the manufacture must, in addition to the above requirements, be capable of bending sharply to a right angle, at a working heat, without showing any signs of fracture.

All rivet iron must be tough and soft, and pieces of the full diameter of the rivet must be capable of bending until the sides are in close contact, without showing fracture on the convex side of the curve.

Pins of four and a half inches diameter or less may be rolled iron, but those of greater diameter must be forged.

Workmanship.—All workmanship must be first class; all abutting surfaces, except flanges of plate girders, must be planed or turned so as to insure even bearings, taking light cuts so as not to injure the end fibers of the piece, and must be protected by white-lead and tallow. Abutting members must be brought into close and forcible contact when fitted with splice-plates, and the rivet holes reamed in position before leaving the works, the plates being marked so as to go in the same position in erecting.

Generally the use of bolts instead of rivets will not be permitted, unless they are turned conical and the holes reamed to fit them.

Rollers must be turned and roller-beds planed.

Rivet holes must be carefully spaced and punched, and must in all cases be reamed to fit, where they do not come truly and accurately opposite, without the aid of drift pins. Rivets must completely fill the holes and have full heads, and be countersunk when so required, and machine driven wherever possible.

Compression members must be straight and free from kinks or buckles in the finished piece.

All pin-holes in pieces which are not adjustable for length must be accurately bored at right angles to the axis unless otherwise shown in the drawings, and no variation of more than one-sixty-fourth of an inch will be allowed in the length between centers of pin-holes. Eye-bars must be perfectly straight before boring; the holes must be in the center of the head, and on the center lines of the bar. Whenever links are to be packed more than one-eighth of an inch to the foot of their length out of parallel with the axis of the structure, they must be bent with a gentle curve until the head stands at right angles to the pin in their intended position before being bored, suitable blocking pieces being used to keep them in proper position during the operation of boring. All pieces must be at equal temperatures when bored, and those belonging to the same panel, when placed in a pile, must allow the pin at each end to pass through at the same time without forcing. Pins must be carefully turned, perfectly finished and straight, and when driven in must have pilot nut to preserve threads. No variation of more than one-thirty-second of an inch will be allowed between diameter of pin and pin-hole. In the case of bolts, a variation of one-sixteenth of an inch will be allowed between diameter of bolt and hole. Thickening washers must be used whenever required to make the joints snug and tight.

All iron must receive one coat of raw linseed oil as soon as received at the works, and a coat of approved red oxide of iron before leaving the works. All inaccessible surfaces are to be painted with one heavy coat of red oxide of iron in pure linseed oil. All iron to be scraped clean from the scale before painting.

The whole of the construction to be first-class work, and in strict accordance with the drawings and these specifications. In the case of sub-contractors, the specifications are fully binding on them in every respect, and free access and information is to be given by them for thorough inspection of material and workmanship, and all required test-pieces, etc., properly shaped, are to be provided as may be requested, without charge. All shipments of material not properly inspected and passed are at the risk of the contractor.

In all cases figures are to be taken in preference to any measurements by scale.

No alterations are to be made unless authorized by the engineers.

DISCUSSION.

Discussion by Theodore Cooper, W. Howard White, G. Bouscaren, William H. Burr, S. W. Robinson, Geo. L. Vose, J. B. Davis, Geo. F. Swain, Mace Moulton, A. P. Boller, Mansfield Merriman, James G. Dagron, William Sellers, Edwin Thacher, George H. Pegram, C. C. Schneider, T. C. Clarke, and Joseph M. Wilson.

Theodore Cooper, M. Am. Soc. C. E.—The purpose of presenting this paper before the Society was, undoubtedly, to obtain the benefit of criticism from bridge experts in regard to the accompanying specifications. This paper is one on which there is a decided difference of opinion, and I hope there will be a full discussion upon the subject. My own belief and practice is totally at variance with many of the views herein advocated by the author.

For obtaining the unit strains to be used in proportioning the various members and details of a bridge, the author has introduced into his specifications the form of formulas advocated by Launhardt and Weyrauch as deduced by them from the consideration of Wöhler's experiments.

Of the value of these experiments of Wöhler there is no question. They are the most interesting set of experiments ever made upon the strength of materials, being made in a manner to tax the material in a like manner to that of actual use. The results are remarkable and deeply interesting. They are, however, too limited to satisfy our desires. Theorists have endeavored to fill the want, by deducing formulas from these limited experiments to cover all possible cases in practice. Such are the formulas of Launhardt and Weyrauch. They have been before the profession for some years, and have been pretty thoroughly discussed, notably between the French and German engineers for the past five years.

At the best, they simply represent the breaking strains; whereas, it is now generally accepted, that for permanently safe structures we must have regard solely to the permanent elastic condition of every member.

Wöhler's experiments show very conclusively that while good iron, strained as usual in our testing machines, may have a certain range of elasticity from a zero of strain, this range of elasticity only holds for other kinds of straining by measuring it from the initial condition of the test piece. For example, for material having a range of elasticity of 34 000 pounds per square inch, when measured from zero the range will only extend from — 17 000 to + 17 000 when the material is subject to alternating strains. This is not represented by the above-mentioned theoretical formulas. They cannot therefore be accepted in their full significance. Hence they should not be used as the author uses them. I also object to their use for other reasons.

First.—They leave in too indefinite a shape the definite instructions which should be put in general specifications for the use of those who are to bid upon any work; leaving too much room for varying interpretations by different designers, thus preventing a just and fair competition.

Second.—They tend to obliterate the judgment and responsibility of the engineer, and to place the same upon the contractor, where it does not belong.

The author in like manner treats the question of impact, and, as before, I object to the method. We do not know the effects of impact upon our structures and their individual members with sufficient definiteness to formulate any law, nor do I think we can define a required percentage for the different members in such a manner as to bring a number of different proposals to an equal and just comparison. The form and construction of any member has also a large influence upon its ability to resist impact. A built member formed by riveting a number of pieces together cannot resist such action as well as one homogeneous body, nor will a short suspender as well as a long one.

In all such matters we must be guided rather by experience than theory; our judgment can be guided by the experience of others and also by due consideration of even imperfect theories such as Launhardt's formulas and the various allowances for impact. The results should then be clearly and definitely stated in our specifications.

My own specifications for bridges, which are at least in as extensive use as any other in the United States, are based upon this belief. The unit strain to be allowed upon each kind of member is definitely stated, all allowances being given due consideration before selecting the same.

Another merit of this method is, that the engineer can so assess the unit strains upon undesirable forms as to favor what he believes to be the best practice.

I believe I can claim to have drawn the first bridge specification (Erie Railroad Bridge Specifications, 1879) in which there was no recognition of the so-called factor of safety or "factor of ignorance." This led necessarily to the definite selection of the unit strains for each particular case. The results of my practice and observation since have satisfied me of the desirability of thus definitely defining all unit strains without possibility of misconstruction.

For identically similar reasons I object to the specifying of the unit force as so much per square foot of the exposed surface, and the determination of the lateral stability from such data. It is not correct, and is too indefinite. Lateral strains are due both to wind and the oscillations of the trains. With our usually accepted practice, it is fully as correct and far more definite to specify a fixed amount of lateral force per lineal foot of the loaded and unloaded chords, based upon the results of accumulated experience. I have endeavored to make clear in a general way my ideas as to the principles upon which specifications should be based.

There are many matters of detail in the specification submitted which I could not approve, but it will not be necessary to enter upon these, as they are clear to any one upon comparison with my own specifications. I regret that the author has not expressed his reasons for the following clause: "Girders formed with web plate and angles only, having no upper flange plate proper, will not be allowed." This is so contrary to the practice of every other designer, that it should have a reason. My own practice is never to use any flange plates if angles can be obtained heavy enough to give the required sectional area.

The author in his paper refers to an article of mine published some eight years ago, on the principles governing the thickness of the web pitch of rivets and stiffeners in plate girders. He is wrong in his conclusions therefrom, that I would "place stiffeners 2½ feet apart" in a girder 60 inches deep and 60 feet long.

While the principles are correct from which he deduces this result, I would never in my practice use it to any such extreme. I have seen girders of this kind, but they were made by those who do not understand the principles of designing a plate girder, in my opinion. The

web has a certain duty to perform in transferring the shearing strains, both vertical and horizontal. The horizontal shear between the web plate and the flange angles determines the number of rivets at each part of the girder. Practical considerations determine the number of rivets that can be placed within a definite limit of the angles (about 3-inch pitch for a single row of rivets and about $2\frac{1}{4}$ in double rows). The bearing pressure on the rivets is usually limited to 12 000 pounds per square inch. With rivets then at 3-inch pitch the horizontal shear is limited to 4 000 pounds per square inch of web. The vertical shear is equal to the horizontal shear.

Using a shear of 4 000 pounds, our web plate would be about $\frac{3}{8}$ thick, which would be abundant if there was not a liability for the web to crumple under this amount of compression. This may be overcome in one of three ways, or a combination of the three:

First.—Increase the thickness of the web to the $\frac{3}{4}$-inch thickness, as given by the author.

Second.—Add stiffeners at intervals (my own practice is at intervals about equal to the depth of girder) until a point nearer the center of the girder is reached, where the original web of $\frac{3}{8}$ is stiff enough to resist the shear at that point.

Third.—Reduce the depth of the girder (not desirable in the example given).

Fourth.—Use a thickness of web intermediate between the two extremes with a less amount of stiffeners.

For each case in practice the designer must select the method giving the most economical results, including cost of manufacturing. Frequent cases have come to my notice where the stiffeners added to the webs have contained enough material to have made the web throughout of the maximum thickness, and have not only saved the workmanship put upon the stiffeners, but placed the metal in the web where it would have added considerably to the strength of the girder (though we do not usually allow it to be estimated).

While Mr. Wilson's idea, that many girders act as Pratt trusses, is perhaps correct, I do not consider it good practice to design the girders upon this idea.

When we look back only a few years, and see how crude were the specifications upon which so many of our older bridges were built, it is a wonder the work was done as well as it was, and little wonder that

they are now proving their defects under the present increased loadings.

These specifications then consisted essentially of "bridge to be of such a length; rolling-load, 2 000 to 2 500 pounds per lineal foot; and a factor of safety of 5; workmanship, first-class."

It is too often the case now to see specifications which are simply compilations from other specifications (frequently good ones), but made in such a manner that they are contradictory and unintelligible, clauses being omitted, because not understood perhaps, which modify or complete other clauses which are selected. I have read Mr. Wilson's specifications, and I think my criticisms are based upon a correct understanding of them.

Mr. Wilson's experience has been a limited one in one sense. I do not say this to reflect upon Mr. Wilson, for we all recognize his ability and his standing in the profession. But as engineer of bridges for the Pennsylvania Railroad, his designs have been built by a limited number of builders, and those of the best class, and he has also been largely the designer of the works he has constructed. He has not had that contact with inferior builders necessary to discover how ingeniously they can take advantage of the weak points of a specification.

A specification upon one's own designs and confined to only the best class of manufacturers, need not be as definite or as strong as one for general bidding, or subject to the interpretation of other engineers.

An experience of a wide range has led me to provide for such cases, and thus to introduce clauses which are looked upon as unnecessary by those who have not had the same experience.

Many a contract has been decided in favor of a particular bidder who was sharp enough to take advantage of some clause badly expressed, or some requirement *expected*, but not definitely specified.

It is the engineer's duty to remove these questions of doubt, remove theories and formulas subject to misinterpretation, and state definitely and fully what he wants, so that the reputable and conscientious contractors have a fair and open field for competition against those who are less scrupulous or intelligent.

First-class builders like a strong, clear and full specification, intelligently interpreted.

W. Howard White, M. Am. Soc. C. E.—Mr. Wilson prohibits the allowance of web in calculating plate-girder flanges. I suppose this must be for convenience, or does he in constructing his theory of girder-web action abandon entirely the old method of calculation, as a beam with notches cut out of the sides?

Does Mr. Wilson extend his prohibition of girders without top plates over angles to floor beams whose top flanges are well held in line by stringers or floor plating over them, and if so, why?

Is he not a little rigorous in exacting flange-joint covers in all cases in compression? Such plates would give rather patchy looking work in long tower posts. Would not planed and well butted, and carefully broken joints do nearly as well constructively, and give neater work? I must take exception to his width between track-tie centers giving clear spaces of ten inches.

To my mind these spaces should be made as small as practicable. The limit should be such width as will allow of examination of the structure immediately under the track, such as rivets and laterals, without being slung below, and it should also not be so narrow as to risk holding coals dropped from fire-boxes in passing. I adhere to the 5 inches which I have previously advocated for clear space in bridge floors, and I have also seen no reason, with lapse of time, for abandoning my preference for inside guards where only one kind are used.

Mr. Wilson places the limit of pressure on masonry, 300 pounds per square inch, very low. This gives only 21½ tons per square foot, a weight which fairly gook brick-work can carry, and any good building stone will carry five to twenty-five times as much. The objection to such a low limit is the excessive size of bed-plates required, particularly in high iron-pier bearings.

Before seeing Mr. Wilson's paper I had intended making some observations on wind strains in continuation of the discussions in previous years on the papers by Messrs. Ashbel Welch, C. Shaler Smith, and F. Collingwood, Members Am. Soc. C. E., and as they had hardly seemed of sufficient value or importance to put forward as a separate paper, I am glad to hitch on to Mr. Wilson's more powerful train.

In considering wind strains there is a good deal of latitude as to how much wind pressure should be allowed, and as to the nature of the distribution of the same.

For instance, we in the United States nominally allow as a rule 30

pounds per square foot of exposed surface as a maximum, while the new Forth Bridge is being built on a 56-pound basis, and in Germany the practice is very various.

The first question which presents itself is, What is to be considered as exposed surface?

Probably, on inquiry of bridge engineers, nearly as many different statements of the manner of making this allowance would be got as there were answers. They would, however, differ mainly in such matters as allowance for smaller surfaces, such as laterals and sway braces, rails and adjoining guard timbers, which would not make a very material difference in the total wind pressure.

The allowance made at the Forth Bridge, as stated by Mr. Baker in his address before the British Association in 1884 (see *Engineer*, 29th August, 1884), is to be "fifty-six pounds per square foot, striking the whole or any part of the bridge at any angle with the horizon, and acting squarely or obliquely upon an area equivalent to twice the plane surface of the front girders, with the deduction of 50 per cent. in the case of tubes."

Mr. Baker used in his experiments a very ingenious arrangement for ascertaining the area of flat surface equivalent to the surface of his bridge members. This, as described in his address, consisted of a horizontal cross-bar suspended by a string at its middle point. A model of the portion of the bridge to be tested is placed at one end of the bar, and an adjustable flat surface at the other. On swinging the pendulum so arranged with the cross-bar at right angles to the plane of oscillation, if the surfaces balanced were not equivalent in resistance, one or the other would advance. The difficulty with this process seems to be to ascertain the center of pressure of the model with accuracy. How this was done was not stated. Possibly it might be determined exactly, but rather tediously, thus: First, assuming the center of pressure, clamp the model on the bar at a given distance from the string on a pin through the assumed center. Balance the model for pressure by a flat surface, with its center of pressure at same distance from the string as the pin. Then turn model partly round on pin and see if the result agrees with the previous one, adjusting accordingly until the model in any position with regard to the center, balances the flat surface. Mr. Baker's results are interesting, and in some respects surprising, inasmuch as he found that two plates corresponding to girders, connected by a floor plate at the bottom, gave only

90 per cent. of the resistance of a single plate. It is difficult to see how this can be. Mr. Baker's results on disks placed one behind the other, show the small resistance produced by our chord-bar arrangement as compared with European systems. He found that with two disks 1.5 diameters apart, the resistance to wind was one and a quarter times that of a single disk, and other disks could be introduced at will without increasing the resistance. In point of fact it would seem probable, from his results on the effect of connection plates, that intermediate plates would diminish rather than increase the pressure on two plates at a given distance apart, by preventing free access of the wind to the surface of pressure. Further particulars of the results of the experiments can be found in the above-mentioned paper.

A further question as to chord-bars is the effect of wind pressure on the outer one of a set in long panels. Where a single bar receives the pressure, and takes up the chord strain, it will yield laterally until the resistance to transverse strain, and its action as a catenary, together balance the rod pressure. The increase of tension in the stretched side of the bar is its only increase of fiber strain, since the aggregate tensile strain in the bar must remain the same—namely, that due to its action as a tension member of the truss—whether it be straight or bent by wind.

If, however, the outer bar be one of a set, as is usually the case, protecting the other bars from deflection by wind, but forced to extend itself in order to take up the transverse pressure of the wind, which would otherwise cause infinite tension in this bar, the question is much more complicated, and the strain produced more serious in amount—a very important factor, in fact, in determining what the maximum strain in the chord really is.

If one assumes that the outer chord-bar of a panel 25 feet long is deflected by the wind into contact with the next bar, say 1¼ inches; in order that the bar may be held in this position by catenary action, a certain longitudinal strain is required, producing a definite elongation. A rough calculation of the effect of a 30-pound per square foot wind will show that the length of the arc formed by the bar with the deflection assumed, has not increment enough over the length of the straight bar to produce this elongation. It follows, therefore, that the bar, with such a wind, must rest against the next one, which, in turn would be deflected; and the result of this action would be a slight deflection of the pins were it not that the same action is taking place in the next panel. The whole problem thus

becomes excessively complicated. A solution of it, thoroughly in accordance with the spirit of our bridge building—and, indeed, with the spirit of our mechanical art in general, which aims at the avoidance of complicated situations as to strains or mechanism—would be the insertion of stops between the chord-bars. These might be arranged to project downward from a horizontal head lying on the chord. Or they might consist of a series of plates, clamped together by a bolt at top and bottom, and imparting a desirable stiffness to the chord, though the solution would be objectionable in appearance.

Before going further into the details of wind-pressure effect, I will return once more to the amount of allowance to be made. The absolute amount assumed per cubic foot is, in any event, somewhat arbitrary.

It would not be worth while, if indeed possible, to build bridges strong enough to resist the hurricanes they are occasionally exposed to in the United States. Thirty pounds probably covers all tempests which are not really phenomenal, and it is well to build bridges strong enough to resist such a wind pressure when unloaded.

The question, then, is: What pressure ought they to resist when otherwise fully loaded? A loaded box car will stand upon the tracks with over 30 pounds per square foot of wind pressure, but a man cannot remain upon his feet in such a wind, even though his body is horizontal, and his legs at an angle of sixty degrees with the ground. About twenty-six pounds is the limit for keeping his feet. Now, though a train might be found on a bridge when such a pressure occurred, it is hardly possible that an engine-driver would take his train on to a bridge while such a wind is blowing. Occasional gusts of such force would be resisted somewhat by the inertia of the bridge itself, and might be allowed to produce an effect on the loaded chords up to the elastic limit. Something, however, should be allowed in chord strains for a surplus strain due to wind pressure in excess of load strains, though, strangely enough, it is often not done, the bridge being proportioned for wind pressure in the laterals without any allowance at all in the chords.* It will be evident from the above, that while indorsing thirty pounds per

* Mr. Wilson neatly covers this point by providing that "in case the maximum stresses in the chords of bridges or flanges of floor girders due to wind and centrifugal force (the chords and lateral bracing being considered as a truss lying on its side, added to the maximum stresses in the chords or flanges due to vertical loading) shall exceed the above-mentioned limits of 15 000 and 12 000 pounds per square inch in tension, and 12 000 pounds per square inch in compression properly reduced, additions must be made to such chords or flanges until such limits are not exceeded."

square foot as a basis for laterals, I would proportion the chords for a wind pressure in excess of the strain produced by the load of something considerably less than thirty pounds—say twenty pounds. Practically all wooden bridges in the United States, and a large part of the iron bridges of the United States and Europe, are built in such a way that the wind pressures in the top chord are for the most part carried down to the lower chords by transversely straining the posts, and thence, through the lower laterals, to the abutments. At the same time these bridges are not proportioned against wind on any such basis.

Where the bridge portal is not strong enough to take up the transverse pressure produced by wind in the top-chord system, it is obvious that there is no advantage in increasing the sizes of the top diagonals toward the ends, since there is nothing for them to bear against. The laterals (diagonals) in such cases are merely an assistance in holding the top chord in line, and they can, in the case of a deck bridge thoroughly sway-braced, be perfectly well omitted, if it is desirable to do so for any reason, such as an exigency of floor construction.

In truss bridges, without strong portals, the rocking motion of the wind causes an increased percentage of load on the lee chord. If P is the wind pressure per lineal foot of bridge; h the height of its point of application above the chord center; a the width between chord centers; and W the weight per lineal foot of the load on each truss, the increased percentage of load on the lee chord is represented by $\frac{Ph}{Wa}$. This increase of load is to be treated exactly as ordinary load, since it is an absolutely downward pressure at the post foot, and independent of the horizontal effect of wind pressure, the whole of which must in such cases be taken up by the bottom chord.

If any one has any doubt as to the double increase of strain on the lower chord arising in this way, a simple experiment will, I think, dispel such. Take a block of wood, of which the length and width are proportional to the height and width of a truss, while the thickness of the block stands for a panel length or any other unit, and stand this on a table. The friction on the table then takes the place of the lower chord lateral trussing. If we attach a weight by a thread to the base of the block, pulling it transversely to the bridge section, we have a representation of the part of the wind pressure which is borne directly by the lower chord. A weight similarly connected to the top of the

block, and pulled in the same horizontal direction, represents top-chord wind pressure. If the weights are made sufficient to just not slip the block on the table, it will be found that it is immaterial in the effect whether both threads are attached to the bottom, or one to the top and the other to the bottom. It is therefore evident that when the strings are in the latter position, representing a bridge without top laterals, or a strong portal for them to abut against, the whole wind pressure is resisted by the bottom laterals, as was to be expected. But in this position it is also evident that the lee edge of the block gets an increase of load, which was also to be expected; but in view of the tremendous increase of strain thus brought upon the lee lower chord, the experiment is not uninteresting.

When this transfer of top-chord wind pressure to the lower lateral system is effected by short braces in the upper corners of the cross-section, the increased load is be reckoned into the load on the top of posts, and an ugly transverse strain in the posts is caused, which must be duly provided for. For this reason the more recent practice of heavy portals carrying the entire top-chord strain, leaving the post free from transverse pressure, is decidedly the better one, and will be found so also in deck bridges in a majority of cases.

Sway braces, so called, are, however, highly desirable in deck bridges; and in through bridges where the height is sufficient to admit of an overhead panel with horizontal strut top and bottom, and diagonal ties; first, to reduce the free length of the posts; secondly, in double-track bridges to assist in distributing the load over both trusses; and thirdly, to serve the same purpose in single-track bridges in high winds, when the live load is unequally divided upon the two trusses, through the effect of the wind moment on the train.

In deck bridges, where both reduction of post length and distribution of load is needed, sway bracing must, of course, be double-decked.

It will be evident that where inclined portals are used, we save, in addition to other savings, the top-chord strain over the whole length of chord due to wind pressure of one panel, but we throw an additional chord strain into the lower chord through the downward and longitudinal thrust of the lee posts from top lateral action.

Too great care cannot be given to the avoidance of transverse strains from wind-bracing connections, against which Mr. Wilson provides in a general way.

The strains that roll up in these members towards the ends of a large bridge, applied at an intersection with a floor beam or cross-strut center a few inches only from the chord, gives rise to tremendous horizontal moments of transverse strains on members which are often but little calculated for receiving it in that direction.

A corollary of this action is the twisting action produced by it on the feet of posts, and the consequent increase of tension on inside chord-bars.

Much study may be profitably expended to pass the centers of lateral ties as nearly as may be through the intersections of floor-beams or struts with the centers of chords, and in the same plane with the latter.

In this matter of intersections on center lines there is a good deal of slackness in girder practice—witness the lattice girders on the New York elevated roads, where the diagonal centers generally intersect the neutral axis of the chords at least six inches apart, manifestly causing very serious transverse strains in the latter members.

An interesting subject for any one mathematically inclined, will be found in determining what strains come upon the upper lateral rods, and what increase of strain on the verticals of a covered Howe bridge through the tendency of the wind pressure to rock the web system over into the plane of the floor. It is evident that this tendency has to be resisted: first, by increased tension in the vertical rods in preventing the extension of length of braces by rocking over on their lee corners; and secondly, by a bending strain in the top lateral rods at the points where they pass into the chords. This, of course, only where (as is almost always the case, however) these bridges have no sway bracing worth speaking of.

In view of the increased strain thrown upon the lower chords of bridges of this class by their mode of construction—first, by the transfer of the whole wind strain to the lower chord; and second, by the larger proportion of the load thrown upon the lee truss by the rolling moment of the wind—it is only surprising that the chronic weakening almost always found in these lower chords is not greater than it is.

This would be largely diminished by the adoption of portals framed to take up the lateral strains from the top chord without passing them through the lower chord.

In conclusion, I would like to call attention to discrepancies fre-

quently found in proportioning roofs, where allowance is made for 30 pounds, or even more, of wind, when the walls of the structures on which the roofs are placed, such as detached shops, railroad stations, etc., would not stand anything like such a pressure.

G. Bouscaren, M. Am. Soc. C. E.—*First.*—The principal feature of Mr. Wilson's specifications is the general application of Launhardt's formula for the determination of the allowable stress in all classes of bridge members. As this is a radical departure from the time-honored rule of the "ultimate resistance" or "limit of elasticity" reduced by a "factor of safety," it is interesting to inquire as to the title of the new rule to such an important succession.

Launhardt's formulas are the outcrop of Wöhler's and Spangenberg's experiments, made principally on tension and bending. The value of u for compression is not yet determined by experiment. It is simply assumed that this value is the same as for tension.

No experiments have been made for alternate tension and compression, except where the opposite stresses are equal. The formula for alternate stresses of opposite signs is deduced entirely by a process of reasoning from the results obtained in this particular case. The influences of rapidity of repetition, rapidity of increase of stress, and duration of individual stress have not been investigated.

The formulas do not take into account the influence of shapes and sizes of individual members. We all know, for instance, that the same working stress should not be allowed on a short suspension link as on a long tie rod; nor on a large tension member as on a rod of small diameter. Presumably repetition of stress must also give very different results for hard than for soft ductile metal, whether iron or steel.

In view of these facts, it would seem that the general application of Launhardt's formulas to the variety—in quality, shape, sizes and conditions—of bridge members is premature, and that adherence to the old rule, for the present at least, is the safer plan.

All that we can legitimately deduce now from the experiments of the German engineers, is that a strain within one-half of the elastic limit is allowable for any number of repetitions of the same kind; this is, in fact, nothing more than a confirmation of the old rule, with a factor of safety of one-fourth, inasmuch as the elastic limit is very nearly one-half of the ultimate resistance. This factor is generally adopted for all

parts of structures subject only to static loads. It is further reduced to one-fifth for structures, such as bridges, exposed to vibrations and contingencies beyond the reach of exact analysis, where the action of the elements, imperfect adjustment, and the secondary strains due to the deformation of the structure under load, all come in for their quota as disturbing elements.

Further than this, the old rule has the sanction of experience. This is certainly an important point in its favor.

I do not know of any instance where a bridge properly designed and proportioned, with a factor of safety of one-fifth, has failed, by reason of weakness, under the load it was intended to carry. In the present state of our knowledge on the subject, the general application of Launhardt's formula is in the nature of an experiment.

It is true that all advances in engineering are the fruits of experiments. But where safety is the prime consideration, it would seem that the most suitable place for experiments is the shop, and that the teachings thereof should only be carried into practice when shorn of their speculative element.

In the particular case of members subject to alternation of tensile and compressive stress, the ordinary factor of safety rule will not apply, as account must be taken of the special weakening effect due to the alternation of stresses of opposite signs. The usual practice of assuming for the working stress on such members the sum of the two maximums, leads in most cases to results evidently too large. Experiments to show the exact measure of the fatigue of the metal in such cases are very desirable; in the absence thereof, I have lately adopted the following rule, inspired by Launhardt's formula, for the determination of the allowable stress per square inch in both directions:

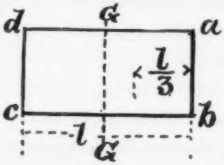
$$\text{For tension, } 10\,000\left(1-\tfrac{1}{2}\frac{s}{S}\right)$$

$$\text{For compression, } R\left(1-\tfrac{1}{2}\frac{s}{S}\right)$$

where s and S are respectively the smallest and largest of the two maximums, regardless of signs, and R the allowable stress for compression, calculated by Rankine's formula with a factor of $\frac{1}{5}$.

Second.—The live load adopted by Mr. Wilson is not likely to be exceeded in the near future, although some roads are, I believe, using consolidation and passenger engines with respectively 26 000 and 45 000 pounds on each pair of drivers.

Third.—The rule requiring the concordance of the line of stress with the line of center of gravity for each member is a very important one. The extent to which members may be overstrained by the unequal distribution of strain, due to the non-observance of this rule in designing details and connections, will be realized, if we remember, for instance, that in the case of a prismatic member of rectangular cross-section of width b and area Ω, the stress per square inch on the edge $a\,b$ is $2\,\frac{S}{\Omega}$, and on the edge $d\;c$, 0 (S being the total strain), when the center of stress is at $\frac{1}{3}$ of the width from $a\,b$; and when the center of stress falls on $a\,b$, then the stress per square inch is



on $a\,b$, $4\,\frac{S}{\Omega}$

on $d\,c$,— $2\,\frac{S}{\Omega}$

Fourth.—With regard to the painting of bridges, there seems to exist some difference of opinion among engineers as to choice between oxide of iron and red lead. I have used both extensively, and give the preference to red lead.

On the Cincinnati Southern Railway, structures painted with three coats of the best quality of iron-clad paint had to be repainted the fourth year; structures painted with red lead in the sixth year. The latter were then in better condition than the former.

The red lead adheres better to the iron, and fails principally by wear and a gradual transformation of the red lead into carbonate, whilst the iron-clad fails by blistering and scaling, which requires the removal of the old material by scraping before a fresh coat can be safely applied. When the cost of this scraping, as well as the shorter life of the iron paint, are taken into consideration, the difference of cost of maintenance with the iron and lead paint is very much reduced, and there remains, in favor of the lead, a greater sense of security, arising from the fact that less damage will occur in case of neglect.

W. H. Burr, M. Am. Soc. C. E.—The fundamental principles of bridge design are very generally recognized at the present time to rest on a philosophical basis of principles, according to which the stresses

developed by exterior loading may in most cases be analyzed with sufficient accuracy for the production of a structure whose limit of resistance is essentially determinate. These excellent specifications demonstrate the feasibility of attaining this result, and show how thoroughly rational structural design has become, even in the two or three points in which it is open to criticism on purely rational grounds; for these very features are in the main avowedly theoretical. It is a matter of surprise, however, that a series of such excellent and minute directions for design should be preceded by an arbitrary approximation regarding the moving load, for it will be observed that after having placed the moving load in such a position that the locomotive drivers are over a cross-girder, the load on the latter shall be considered the head of the train, all preceding load being omitted. The reason for taking some other load than the actual, does not appear.

The methods for computing the absolutely greatest stresses for any system of wheel concentrations are so obvious and so simple, that it is difficult to imagine why they should be displaced by an approximation which may not always result in a safe error.

The recognition of the fatigue of metals, as shown by the tests of Weyrauch and Spangenberg, is certainly justified; but it should be borne in mind that they experimented on prepared test pieces, and not on finished bridge members. This is markedly true in the fatigue of pure compression, although it is probably much less than that of pure tension; in fact it is not an exaggeration to state that almost nothing is yet known regarding the fatigue of pure compression. It would seem, therefore, a little hasty to frame a formula for full-sized compression members from results on tensile specimens. In reality, it is probably most rational (as it is convenient) to fix the minimum working limit for the first counter and increase uniformly to the maximum value for the main tension member nearest the end of the truss, and proceed in the same manner for the compression web members.

The matter of web stiffeners for plate girders is one in which the author has widely missed the rational basis which he has certainly sought. The action of shearing stresses in web plates has been thoroughly understood since the establishment of the mathematical theory of elasticity in solid bodies, at a period so long previous to Professor Rankine's day that he acknowledges his indebtedness to Mr. Lamé. It would be interesting to learn the details of the experiment by which he

concludes "that the web resists by tension and not by compression." If the web of the paper model was made flexible by slitting it at 45 degrees (or any other angle) to the neutral line, it was obviously rendered incapable of resisting by compression, and the experiment would show nothing regarding the point under investigation. It is perfectly well known, and as firmly established as any mathematical truth can be, that with pure transverse shear, as at the neutral surface, equal intensities of tension and compression exist at 45 degrees to it and 90 degrees to each other. The corrugations of a buckled web plate verify these rational conclusions. It is an actual and absolute impossibility, therefore, that a web plate should resist by tension alone.

The tension and compression are inseparable, and must act together. In this fact lies the stiffness of such thin webs as mentioned by Mr. Wilson. The tensile stresses are at right angles to the assumed elementary columns (*i. e.*, to the compressive stresses), and hold them in place at every point of their entire length by a very considerable force, which must be overcome before buckling takes place.

Web plates, therefore, are far stiffer than would appear from the clumsy hypothesis of isolated elementary columns fixed at their ends only. Nevertheless there is a limit beyond which stiffeners must be used in order to retain the web in its own plane, although that limit is analytically indeterminate. But it is practically impossible that the strains or deformations in the stiffeners, rivets and web plate should adjust themselves with such nicety as is necessary to enable a stiffener to take stress as a strut, and it is a matter of great doubt whether such action would take place if such an adjustment were possible. If heavy concentrated loads are on the flanges immediately over the tops of closely-fitted stiffeners, the latter obviously may act as struts, but after the web has once taken the shear, it requires a very extraordinary condition of things to render a stiffener-strut possible. If the web plate can be retained in its plane, its resistance becomes equal to the pure shearing capacity of its tranverse section, and the office of the stiffener is no less important than to produce that result.

Just the frequency with which these stiffeners must be used, and their necessary sections, are yet matters of experience only. They belong to the many still open questions which may be qualitatively, but not quantitatively, answered. The ordinary and conventional rule used by Mr. Wilson for the spacing of the stiffeners is probably as

good as any, after they become necessary, but his sections are quite excessive.

The work to which these specifications, as a whole, lead, is certainly of a most substantial and satisfactory character.

S. W. Robinson, M. Am. Soc. C. E.—In a paper presented by the writer to this Society, on Vibration of Bridges, results of observations with a bridge indicator were given, in which the superadded strain, due to vibrations caused by the non-balanced locomotive drivers, was 28 per cent. of the maximum strain caused by the passing train when statically considered. Likewise an observed superadded strain, due to vibrations caused by the body of the train, was given, which was 50 per cent. of the strain due to the corresponding part of the train statically considered. Also, it was shown that certain diagrams taken gave statical strains for the train itself in excess of like strain due to the locomotive for the same trains.* Also, the observed fact that in practice train loads may exceed engine loads † was mentioned as a reason why the 50 per cent. should be provided for instead of the 28 per cent. in designing bridges where conditions favor vibrations from the train.

The bridge from which these particular per cents. were obtained was a through iron Pratt truss of 141-foot span, and 9 panels of 15⅔ feet each.

The above percentages are due to cumulative action or to a series of conspiring impulses, and therefore of much greater intensity than if produced by a single impulse of the same kind. Also, it should be noted that the percentages stated are not hypothetical ones, but are solid facts of observation read off from the direct and positive autographic records made by the bridge of its own vibratory movements occasioned by the passage of trains. Most of the 13 bridges from which autographic records were obtained, gave results of like kind, though not all of equal degree.

Some foreshadowing of these results was given in the report for 1881 of H. Sabine, Commissioner of Railroads for Ohio; also in Van Nostrand's *Engineering Magazine* and Science Series, and in numerous periodicals

* This statement of strains is made on the supposition that the greater strains are accompanied by the greater deflection.

† By "engine load" is meant the loading upon the bridge, consisting of engine and portion of following train.

by way of republication; but it may be presumed that some bridge specifications framed subsequent to those publications failed to give full credence to the effects whose existence was thus pointed out, for the reason that those effects were not sufficiently determined in degree to warrant it. But as results are now determined beyond question, and of by far too high a degree to ignore, this opportunity of calling attention to them is taken advantage of; but it is with the understanding that the specific results above mentioned were not made public until after the bridge specifications now being discussed were proposed, both being subject matter presented at the Annual Convention of 1885.

No provision is found in the specifications for the kind of dynamic effect above pointed out, except it be in a general way, as with all bridge specifications, by such excessive sections, or by low unit stresses, as is found in some unaccountable way to be necessary. For example: Why should a unit stress of 7 500 pounds be necessary for a member composed of first-class wrought-iron, when the experiments of Wöhler and Spangenburg show that such iron can stand for an indefinite number of repetitions a unit strain of about 30 000 pounds, unless it is to meet some practical demand the foundation for which is hitherto unaccounted for? If good iron will stand unit strains varied from 0 to 30 000 pounds repeated millions of times and yet be sound, where all strain is accounted for; then in bridges, if all strains are accounted for, why cannot the iron there work up to the same limits, or at least to two-thirds of it, something being allowed to make safety doubly sure? But a 20 000-pound unit stress is an unheard of permissible stress for a bridge in recognized good engineering practice.

But notwithstanding these extravagantly low unit stresses in present practice, iron bridges do sometimes go down, while the cause is passed as mysterious and unaccountable. How can it be otherwise than a stigma of disgrace upon the engineering profession that this hundred-fold extravagance in use of iron exists to this day. Can it be otherwise than conceded by any rational mind that if all strains are accounted for, the working stresses of iron members may be carried up to at least two-thirds that which it is well known the iron will stand for repetitions well nigh infinite in number?

But, again, where unaccounted for strains exist, cannot they exist to greater intensity in some individual cases than in others without our knowledge, necessitating for a practical solution of the problem, the

adoption of less and less unit stresses, until such a gross allowance comes to be blindly made as to cover up all ignorance of strains. As long as one bridge in a thousand is subject to a 50 or 100 per cent. unaccountable strain, all must be so designed as to provide against that strain, and yet the unfortunate bridge whose unknown strain may reach a limit higher than tentative practice has made provision for, must go down; instances of which are well known. When an iron bridge goes down, the allowed working stress will go down, and so practice has continued until the present extravagant limit is reached of one-fourth that for infinite endurance according to definite experimental tests.

But fortunately a part of this hitherto undetermined strain has been made known through the investigation on bridge vibration, thanks to the persistence of Commissioner Sabine, of Ohio, for pushing the matter to the definite experimental results, already stated above, of 28 and 50 per cent. for the head of a train and body of a train respectively. Accounting for this latter percentage in bridges of 150-foot span, the allowed working stress may be raised about 30 per cent. for the same dead load, but in designing the bridge for a given live load, this percentage of increased unit stress will lighten the bridge and give occasion for a still further reduction of weight and corresponding decrease of strain upon the members. Suppose A to be the total possible strain likely to occur, and that the bridge will just be able to meet it on the basis of a 30 000-pound unit stress, then, with an allowed unit stress of 10 000 pounds, only one-third of the possible strain is accounted for and two-thirds not. But if 30 per cent. of the heretofore known strain be made known and added, it will be $\frac{1}{3} A + \frac{1}{3} A \, .30 = .43\, A$, or 43 per cent. of the possible strain is then known and can be accounted for, and accordingly the working stress can be carried up to 43 per cent. of the 30 000 pounds = 13 000 pounds, the additional 3 000 pounds being 30 per cent. of the 10 000 pounds, as above stated. Hence if all the now known strains be provided for, the allowed unit stress may be raised about 30 per cent. above values heretofore used with equal safety. For simplicity of illustration, these percentages have been figured on the basis of the formerly accepted unit stress, 10 000 pounds. Under the refinements of Wöhler's laws the figures would be slightly different.

But it may be said that we gain nothing by raising the stress unit along with the strain unit. If no effort is made to avoid the useless and unnecessary 30 per cent. strain due to cumulative vibration it is true,

but by so designing the bridge as to relieve it of this as much as possible, the increased unit strength may be employed with usual safety, and with a 30 per cent. saving in iron for main trusses.

To avoid the above mentioned 30 per cent. strain, cumulative vibration must be stopped, and this important step only requires that the panels of the bridge be in discordant relation to the half-car lengths, as pointed out in the paper on Vibration of Bridges. As no cumulative vibration was discovered for passenger trains, aside from the engine, we have only to consider the relation of panel lengths with freight cars. As the latter cars range in length between about 29 and 33 feet, then bridge panels should not be very near 15 or 16 feet, nor 7 or 8 feet. Probably 9 to 13 and 17 are the best panel lengths, as comparatively few coincidences occur with lengths between 9 and 13 or 17 and 20.

In deciding upon the panel lengths of any bridge, the prevailing length of freight cars, coupling to coupling, should be known. Then the half-car lengths may be compared with proposed panel lengths thus, for lengths continued in succession—

Half cars...........	0,	15,	30,	45,	60,	75,	90,	105,	120,	135.
Panels..............	0,	13,	26,	39,	52,	65,	78,	91,	104,	117.
"	0,	12,	24,	36,	48,	60,	72,	84,	96,	108.
"	0,	9,	18,	27,	36,	45,	54,	63,	72,	81.

Here we find that for the 13-foot panels the first coincidence after 0 is at about 100 feet from that point, which admits of but one or two coincidences on the bridge. In the 12-foot panel lengths we find coincidences at every fourth panel, and in the 9-foot panels at every third panel; the 13-foot panel being here preferred, and for which it is believed that cumulative vibration of considerable magnitude cannot occur.

If by making panel lengths 13 instead of 15 feet a 30 per cent. strain can be avoided, no one can reasonably deny the advisability of the 13-foot panel. This preference becomes all the more imperative when it is remembered that the cumulative vibration may go far beyond that as yet observed, and reach 100 per cent., or even more, as its possibilities are only limited by length of train, as pointed out in the original paper.

Indeed, beyond the now known 43 per cent., the entire balance of the rarely occurring, unaccounted for 57 per cent. strain above must be due to the same cause. In searching for the cause can we find any other than this one of cumulative vibration, and if we find no other, are we

not warranted in accepting this one as sufficient to account for that balance, when a previously unknown 30 per cent. portion has been actually brought within the range of positive fact from a few weeks observing, and when the possibilities of the same cause reach even beyond the whole balance of 57 per cent.?

In view of the above facts and figures relative to panel length, the necessity of a clause controlling panel length is strongly recommended for both economy and safety, in the present as well as all other railroad-bridge specifications.

By proper panel length and consequent avoidance of the strain caused by cumulative action of the body of the train, there will still be left the cumulative dynamic effect due to the unbalanced locomotive drivers of the above named 28 per cent. But fortunately in this case the possible impulses are determinate in number from the bridge itself, and much more limited than in the former case; and it will be much safer to assume a working limit for this strain than for the former, if indeed it be necessary to do so instead of adopting the advisable alternative of exactly counterbalancing the locomotive mechanism.

In deciding upon a safe limit, it must certainly be taken in excess of that observed within the few weeks observing, even for 150-foot spans, and a little consideration will show that it will vary with the span according to a law the precise expression of which will be considerably complex, dynamics making it about as the square of the span, while practical conditions would probably reduce it to about the first power of spans or a little greater. If it be granted that the possible effect for a 150-foot span is 50 per cent. greater than that observed (and I believe this to be none too large an allowance), the 28 will be raised at once to 42 per cent. of static strain due to live load for 150-foot spans, and for 300-foot spans 84 per cent., or about 28 per cent. for each 100 feet of span. This amounts to determining the live load L, where non-balanced drivers are employed, for use in calculating the bridge strains from the formula.

$$L = L_1 (1 + .0028 S) \qquad (1)$$

where L_1 is the usual live load per foot run, and S the span of bridge. For a span of about 360 feet, this formula doubles the unit live load.

As to allowing for this strain due to cumulative vibration caused by the engine, and also that due to body of train, both in a given case, it seems not to be necessary, from the fact that they do not occur simultaneously, and the diagrams from the bridge indicator show that the vi-

brations caused by the engine hardly extend over into those due to the train. And as the latter, though largest from actual observation, are subdued by a judicious selection of panel length, the former strains only remain to be necessarily provided for in selecting the maximum load for calculation. If these strains be regarded as covering the whole ground of the hitherto unaccounted for strains, then the safe working unit stress may be carried up from the former 10 000 to say 20 000 pounds for the live load, equation (1), and with no hazard for safety.

From these facts and figures, it is believed that the present specifications, as well as all others, should embrace the above formula for determining the live load, as well as the clause for controlling panel length, and that the permissible working unit stress should be carried up accordingly.

As to eliminating the difference $L - L_1$ above in strain calculations, the civil engineer is at the mercy of the mechanical engineer, with little hope of relief, from the fact that at present there seems to be but little disposition on the part of motive-power superintendents to displace the present defective unbalanced mechanism by that which is balanced. But there can nevertheless be no question, that as long as the present objectionable mechanism is in use, the cost of bridge trusses must in consequence be increased by not less than about 18 per cent. for 100 feet spans; 30 per cent. for 200 feet spans; and 36 per cent. for 300 feet spans. In the meantime may all bridge specifications prepare for the advent of balanced locomotive machinery, by so fixing the panel length as to destroy the greatest of the two effects of cumulative vibration.

Relative to an attempt to provide for cumulative vibration strains along with the Launhardt and Weyrauch formulas, a little consideration will show that the formulas will require doctoring in the opposite sense to that practiced by Prof. Wm. Cain for including the impact as due to single impulses of various sorts; whereas, for the so-called impact, the greatest allowance is made for short spans, and vanishes at spans of about 100 to 200 feet. We have found that the cumulative vibration strains due to the engine's non-balance increase with the span; and by casting over the figures, it is seen that up to spans of about 150 feet, the total superadded strains to be provided for as due to both impact and vibration, amount to a nearly constant fraction of 50 per cent. of the strain due to the weight of the train, and more for longer

spans; the cumulative vibration due to the train being here and hereafter in this discussion regarded as avoidable. Up to about 150-foot spans then, the Launhardt formula should be applied to railroad bridges without modification as regards impact, while the provisional percentage is added to the load; and for longer spans the greater percentage. This would restore the coefficient $\frac{1}{2}$, which has been set aside by Prof. Cain, allowing that the $\frac{1}{2}$ generally used in the formula for iron is the best selection. But doubts may be expressed as to this value of the fraction, from the fact that it is much lower than that obtained by making use of all the existing data.

The value $\frac{1}{2}$ appears to have been adopted by Weyrauch with but little discussion, using a tensile strength of 46 800 pounds per square inch, a tenacity which would not be permitted for important tension members. Again, the fraction $\frac{5}{6}$ is offered by Spangenberg for this formula and obtained by Weyrauch, using a tensile strength of 57 160 pounds per square inch; a tenacity that is rather high, but indicative of better quality of iron than the former value. Why is the fraction $\frac{1}{2}$ adopted into the formula by Weyrauch for use by engineers, instead of the $\frac{5}{6}$, unless from timidity for departure from usual previous practice?

If the fraction $\frac{5}{6}$ be regarded as intrinsically better than the $\frac{1}{2}$, then the value 1 adopted into the formula for the Pennsylvania Railroad specifications is not very far from the $\frac{5}{6}$ found for good iron, leaving only $\frac{1}{6}$ instead of the supposed $\frac{1}{2}$ to account for the impact. In view of the value $\frac{5}{6}$, it may be questioned whether a higher value than 1, as $1\frac{1}{3}$, should not be taken in providing for impact.

From the vacillation of Weyrauch and others between the fractions $\frac{1}{2}$ and $\frac{5}{6}$, it seems desirable that some unquestioned value might be fixed. In searching for existing data to base a value upon, very little apparently can be found, and yet that, as meager as it seems, cost 12 years of experimental work by Wöhler, and was confirmed by Spangenberg. Hence the existing data, though scarce, is precious; and it seems hardly reasonable to adopt a constant almost by guess, to stand as the consideration for so vast an expense.

All the results accessible to me are:

	Max. Stress.	Min. Stress.
I.—For Iron—pounds per square inch,	$s = 16\,640$	$s = -16\,640$
	$u = 31\,200$	0
	$a = 41\,600$	$c = 24\,960$
	$t = (52\,000)$	$t = (52\,000)$
II. " " "	$u = 31\,300$	0
	$t = 57\,000$	$t = 57\,000$
III. " " "	$u = 31\,300$	0
	$t = 47\,000$	$t = 47\,000$
IV.—For Steel—pounds per square inch,	$s = 29\,100$	$s = -29\,100$
	$u = 49\,200$	0
	$a = 83\,200$	$c = 36\,400$
V. " " "	$u = 52\,000$	0
	$a = 72\,800$	$c = 26\,000$
	$a = 83\,200$	$c = 41\,600$
	$a = 93\,600$	$c = 62\,400$
	$t = 114\,400$	$t = 114\,400$
VI. " " "	$s = 22\,880$	$s = -22\,880$
	$u = 39\,520$	0

The values in *I* for iron are found in "Fatigue of Metals," by Spangenberg, except the figures in brackets, which have been supplied with the belief that they are about right for t, for the iron in question. The values *II* and *III* are found in Weyrauch, the former being that from which the value $\frac{2}{3}$ was obtained, and the latter the value $\frac{1}{2}$, by Weyrauch, for the Launhardt formula.

The values for steel are found in Spangenberg.

All these data are plotted on Plate XXXVIII A in 2 and 1 respectively, both in 3 (the square dot in 2 between s and u belongs to 3). Curves are carefully drawn through among the points, regardless of formulas, with the object of finding the best obtainable from the data. In. 3, to which 1 and 2 are transferred, we see no very wide difference in general character of curve, both appearing to be parabolas. In plotting the curves, the maximum and minimum stresses are laid off from the zero line as indicated, each series of values being laid off in the order—circle, triangle and square for iron, and square, circle and triangle for steel; the "*I*" through the dots for iron, serving to distinguish iron from steel in 3, which shows the plottings for both iron and steel.

In plotting these curves, the line between c and d is taken at 45 de-

grees for convenience in plotting; c then being the distance from the foot of u, as well as the corresponding height to the 45-degree line.

For the steel, all the data are reduced proportionally to bring them about to the values for iron in order that they may be thus plotted in comparison.

At first it was proposed to adopt one curve for both iron and steel, but the absence of material advantages in this, and the prevailing difference of the curves, determined otherwise.

In examining these plottings and their curves, it would hardly seem necessary for better agreement of curves with points, that the portion between s and u differ from the rest; that is, that these parts follow different curves and laws; and, much less, be discontinuous at u. The plotted points for steel appear a little within the curves drawn at the point u, and it would seem that the curve might be straighter to advantage. But instead of a straighter curve, the present one has been insisted upon, for the reason that it is believed that the curve must not be discontinuous at s with that going to make the compression part of the diagram, and this requires that the tangent to the curve at s be parallel to the 45-degree line. Hence, the full line curves are believed to be the best that can be drawn in without introducing discontinuity at any point.

To show how these curves compare with those of the Launhardt and Weyrauch formulas, points have been determined by calculation from those formulas with their adopted fractions as found in Weyrauch, the points plotted and the curves drawn in with dotted lines for steel and iron at 1 and 2. These curves appear to be sadly wanting in both continuity and agreement with Wöhler's observed data.

In the Pennsylvania Railroad specifications the value of $\frac{u-s}{u} = \frac{1}{2}$, is the same as for the dotted curve between s and u for the curve 2, Plate XXXVIII A; but the adopted value of $\frac{t-u}{u} = 1$ in the same specifications, for the curve between u and t, is such as to place the curve further away in the direction of stress than the dotted line. This causes an excessive convexity from u to s; makes $s = \frac{u}{2}$; swells intermediate values between u and s out of all reason; and introduces a decidedly questionable intersection of the curves at u, or discontinuity.

From these considerations it would seem that the formulas are little better than guesses at the real requirements in the matter, and hence the

full line curves in Plate XXXVIII A have been drawn in as founded, as fully as possible, upon the experimental results of Wöhler.

When the curves were drawn, that for iron was found to be appreciably close to a parabola whose axis is coincident with the zero line of stress, and vertex at a distance at the left of u equal $\frac{3}{4}$ s, while that for steel agrees well with a parabola whose axis is below the zero line. But when diagrams are used for determination of working stresses, there will be no need of formulas.

In the upper part of Plate XXXVIII A, the curve 2 for iron has been laid out along with all the quantities involved in the problem of determining working stresses, so as to indicate to the eye their true relation.

Plate XXXVIII B, for that part of the curve from t to u, is laid out according to the formula of Launhardt, where $\frac{t-u}{u} = 1$, as adopted in the Pennsylvania Railroad specifications. This curve (a parabola) is tangent to a line at 45 degrees at the top end of u, and from that point this tangent is used in this diagram from u to s, instead of the formula of Weyrauch for compression and tension by reason of the objectionable convexity given by that formula, such convexity not only causing discontinuity at u, but giving, as is believed, excessive values to a, between u and s. This diagram is believed preferable to the formulas found in the specifications from which to make out the working stresses when the maximum and minimum strains are determined.

To construct a diagram from which a may be read off, it is necessary that a be given in terms of the ratio of c to a, or of minimum B to maximum B, because practical values of minimum B and maximum B for a bridge member may be found in values a hundred-fold greater than the respective values of c and a, and yet have the same ratio. A good diagram for this purpose is of the form given in Mr. J. M. Wilson's statements respecting the Pennsylvania Railroad specifications.

A diagram of this kind is given in Plate XXXIX, suitable for use in office practice, except that it will be advisable to draw it to a larger scale. This diagram is drawn from values read off from the upper part of Plate XXXVIII A, and hence Plate XXXIX may be regarded as realizing that diagram which works with nearest approach to Wöhler's laws and experimental values. The working stresses a, on the left, are figured with a factor of safety given by dividing the value of u from Plate XXXVIII A, viz.: 31 200 by the corresponding value found at the intersection of the

curves in Plate XXXIX, viz.: 11 400, giving $\frac{31\,200}{11\,400} = 2\frac{3}{4}$ nearly. But where all strains are accounted for, as proposed in this discussion to be done, by taking the panel length such as to destroy cumulative vibration from the body of train, and allowing for like vibration by the engine as proposed in formula (1) above, and also impact, it is probable that a factor of safety of 2 might safely be employed, giving 15 600 to be placed opposite the point of intersection of the curves in Plate XXXIX.*

The unit stress for iron in steady tension or compression, is given by Plate XXXIX at 20 000 pounds, 11 400 pounds when repeated from zero to that figure, and 6 600 pounds for vibration with equal tension and compression.

But in drawing the diagram, any scale of equal parts may be put along the line of working stresses according to the judgment of the engineer, and such scale, whatever it be, will give the working stresses in conformity with Wöhler's results; or it may be more convenient to draw several of the diagonal lines on Plate XXXIX, at the junction of each pair of which, on the left side, may be written the factor of safety.

As a practical confirmation of the views given in this discussion, the instance of a certain through iron Howe truss bridge is mentioned, which stood in service about sixteen years, when though yet sound, it was condemned and renewed, on account of dangerous strains in the lower chord, due to placing the floor-beams on the lower chord, two per panel, and each at a distance of 18 inches from the end of panel. Length of truss, about 105 feet; depth of truss, 20½ feet; panels, 13 feet. Aggregate section of lower chords at end panel, 4 inches breadth by 5 inches depth. This was a very peculiar structure, in which all compression members in the main trusses were of cast-iron, and tension members wrought-iron. See a more complete description in Ohio Railroad Report for 1881, p. 289.

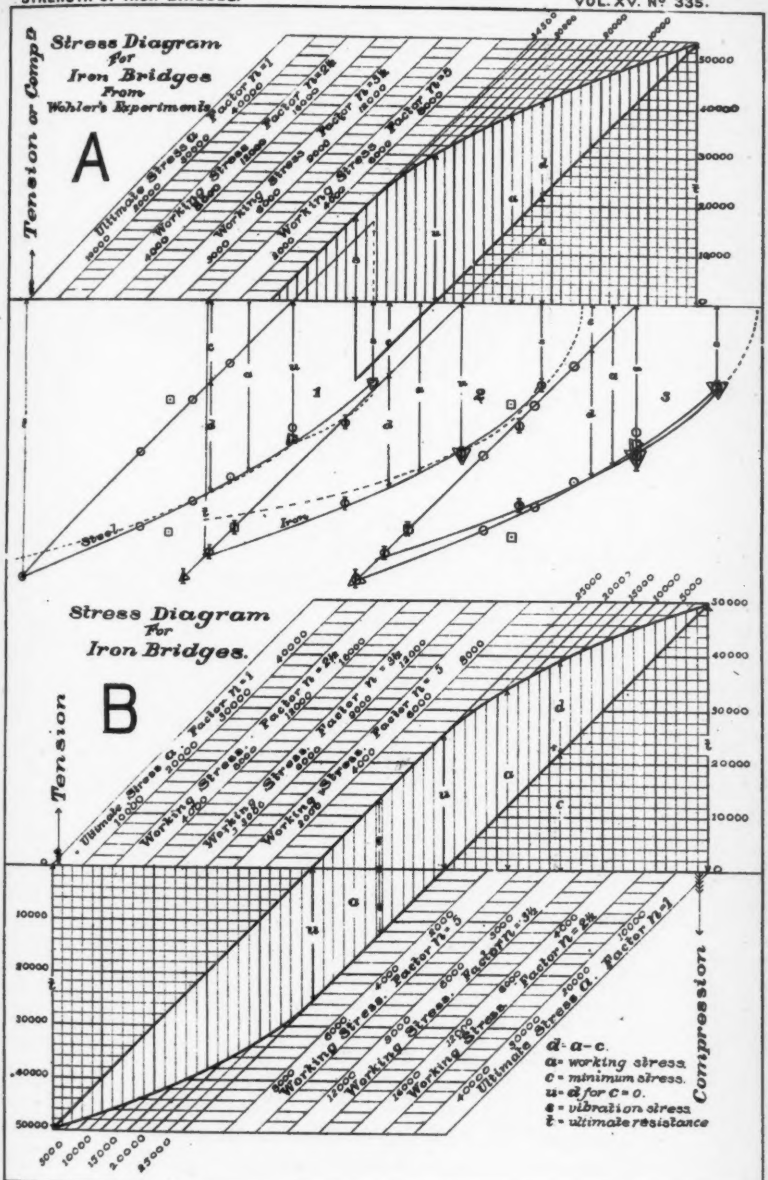
The maximum stress in the lower chords figured up at about 25 000 pounds per square inch for an ordinary train and the dead load. See same report, appendix, p. 44, and Van Nostrand's Science Series, No. 60, p. 175.

If a 10 000-pound unit stress calculated for static load, is as high as is safe in some bridges where unaccountable dynamic strains occur to

* This diagram gives only one pair of lines to the factor of safety 2¾ nearly. But the diagram might be full of these pairs of lines, each to a definite factor of safety, all of which factors, 2, 2½, 3, 3½, etc., could be noted at the junction of the respective pairs of lines, making it convenient to use the diagram for any selected factor of safety.

ROBINSON DISCUSSION ON
STRENGTH OF IRON BRIDGES.

PLATE XXXVIII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. No 335.



carry the stress up to 25 000 or 30 000 pounds, then, if a greater value of calculated strain than 10 000 pounds be found in a particular case like the above, it would seem that the unaccountable part in that case had been low from some cause, so that the sum total of stress should not exceed the 30 000-pound limit.

Hence we inquire whether this bridge was so situated as to be afflicted with cumulative vibration.

As to the panel length, we find the present 13 feet to be an example of the most favorable length, according to figures given above in this discussion; so that it is probable that cumulative vibration never occurred as an effect due to the body of the train.

As to cumulative vibration occasioned by the non-balance of engine drivers, the fact that this bridge was only about 500 feet from a station is against it, as the train would always move so slow as to make this improbable.

These facts, therefore, are in support of the views expressed in the above discussion, viz., that when all strains are accounted for, the working unit stresses may be placed as high as 20 000 or 30 000 pounds with safety.

As a further illustration, the instance of the roof trusses of 180 feet span of a certain Union Depot in the State of Ohio is given. These roof trusses, on inspection, were condemned, as in danger of falling, for strains in the main straining rods greatly in excess of what would in present engineering practice be regarded as admissible.

The strains would be moderate, except for the injudicious and unwarranted splice formed by forging heads upon one side of the rods, which heads were locked one upon another, and banded to prevent slipping off. The bars were rectangular in section. In this way the two bars joined at a splice were out of line, the sides nearest to line offsetting each other by the thickness of the locks. This splice-joint introduced a flexural moment so great, that at the time of the examination the rods near the locks were found so much curved as to be necessarily permanently bent to some extent, because the curvature could not be carried to the extent observed without forcing the iron beyond the elastic limit. The roof has been in service for something like twenty years, with the strains in these rods constantly up about to the elastic limit, as calculation showed; and yet, in spite of all the adverse facts, the roof still stands. The possible strains due to storm and wind dur-

ing the life of the structure are unknown. In some instances snow has been shoveled off to relieve the roof, and it is likely that in a few instances the strains have been 15 to 20 per cent. above the usual and nearly constant strains.

Hence we find here an instance in which strains have been constantly up to nearly 30 000 pounds per square inch for a series of years, far above the 16 000 pounds that might be allowed for a roof, and yet without further signs of failure now than existed ten years ago.

This second instance, therefore, confirms the views stated in this discussion of maximum working stresses, that where all strains are accounted for, the unit working stresses may be carried up to from 20 000 to 25 000 pounds per square inch. This particular roof, however, is not to be regarded as safe, for the reason that there may yet occur such a combination of snow, rain and wind as to cause disaster to the structure by reason of strains greater than those now accounted for.

Finally, to briefly state the points of this discussion, it is seen that I have endeavored to show that this and all railroad bridge specifications should—

First.—Contain a clause placing the panel lengths of the bridge in discordant relation with the half-car lengths.

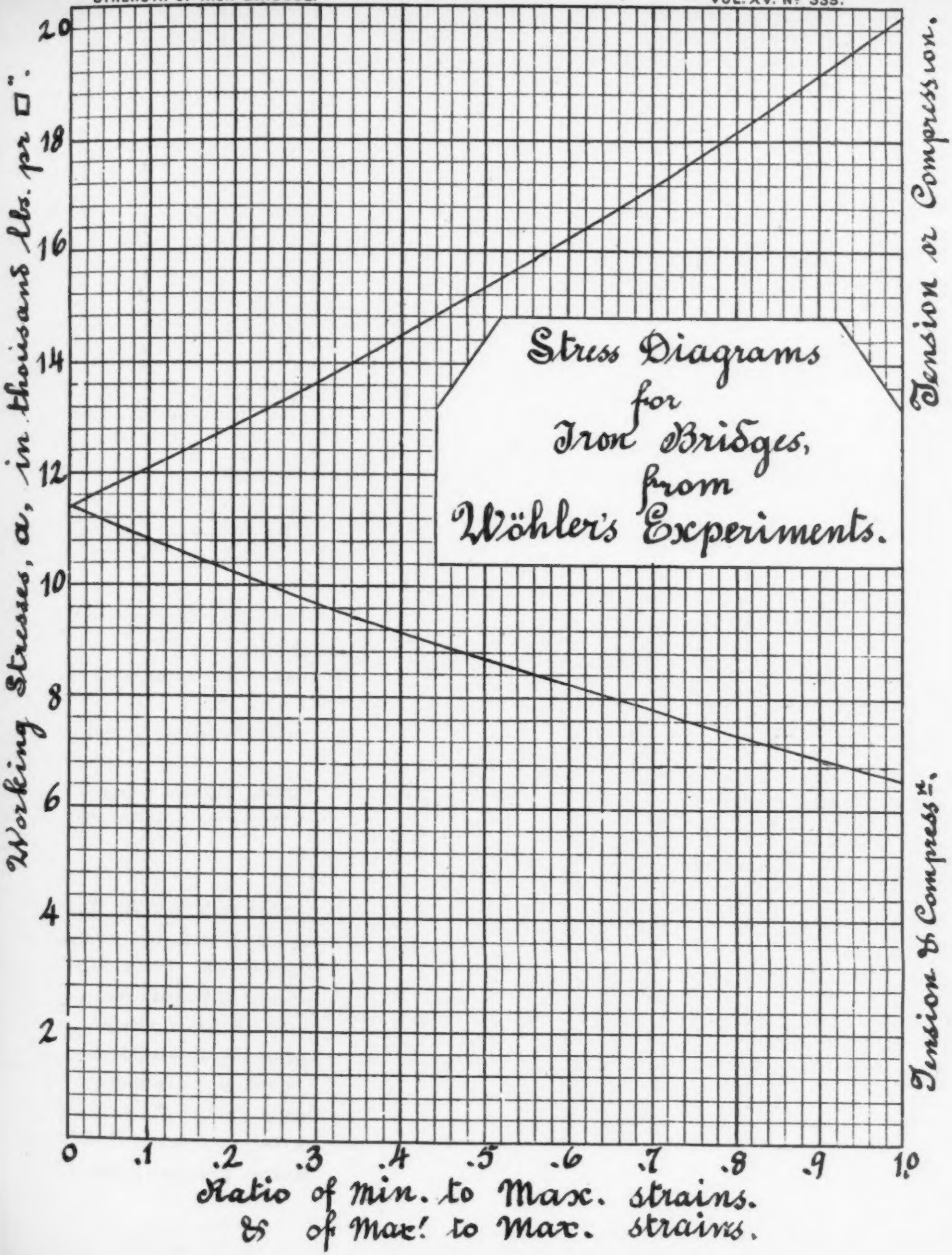
Second.—Provide for cumulative vibration due to unbalanced machinery of the locomotive, as in equation (1).

Third.—Add to *second* the proper allowance for impact in estimating live load.

Fourth.—Employ for determining working stresses a diagram like Plate XXXIX, laid out from Plate XXXVIII A, with a factor of safety of about two, or possibly less.

Prof. George L. Vose.—I have examined with care Mr. Wilson's bridge specifications. There can be no question as to the quality of a bridge built in accordance with such a requirement. Certainly no person could be better fitted by long experience to prepare such a paper than Mr. Wilson. I think he has gone very far in this document to do what is so essential, viz., have everything plainly expressed, and nothing left to be inferred. With a model specification, faithfully carried out, two contractors 1 000 miles apart should produce the same bridge. With many of the specifications in use, two contractors in different places would produce very different structures, and both apparently in

PLATE XXXIX.
TRANS. AM. SOC. CIV. ENG'RS
VOL. XV. Nº 335.



accordance with the same specifications, and both intending to be entirely honest. The more we advance in our knowledge of materials, the harder it seems to be to express our ideas with precision. To say that a bridge should be made with a factor of six throughout, sounds very simple, but I doubt much if any two engineers could agree as to what the factors were in the different parts of a given bridge. I am glad to see that Mr. Wilson does not use this term, which can do little but give false impressions.

To insist in the plainest manner upon the essentials, and to leave no chance for ignorance, dishonesty, or any other form of danger to creep in, is the great point. This Mr. Wilson has certainly done. It may be said that the new specification would not be understood by a board of directors or by the so-called practical man. It is not intended that it should. The specification is for the use of educated engineers who are expert in the principles and the practice of bridge building, as it is only in the hands of such men that the materials of engineering can be safely and economically employed.

J. B. Davis, Jun. Am. Soc. C. E.—I hope S. W. Robinson's column formulas will receive attention in this connection. There is abundant evidence that the end of column formulas is not yet. His are very promising, it seems to me, and are not especially troublesome.

In my own experience in iron truss work, I am accustomed to pack my members, both web and chord, so as to reduce couples about my pins to the shortest of arms, and to bring all my members as nearly as possible into the same planes of stress throughout the whole length of truss. I seek to get these planes continuous throughout the span. To do this I make all my web tension members at least double, which, of course, will be the usual case. I do this however light the work. The result is pairs of members in the same panel. These pairs I require to be matched and marked at the works, and erected by these marks. I also specify a factor of precision for bars of the same pair, and another factor of precision as a limit of extreme deviation between bars of the same specified length in the whole work. Of course this refers to pieces without adjustments. The result, so far, has been quite satisfactory when the trusses are erected and put in "tune," as I call it. I find the bars in the same panel doing the same duty, as nearly as can be ascertained by their apparent tensions.

I consider it very important to study exhaustively how to apply the load to a truss so that no possible expected use of it can fail to convey that load uniformly to its working parts, and distribute it proportionately amongst them. I do not always find it done in existing structures. Likewise I consider the transfer of strains amongst the pieces in just the same light. Everything should be balanced. Of course this is all elementary enough, but is in no danger of being insisted upon too frequently or too forcibly. And this reminds me that a fourteen-year old boy can compute a strain sheet nowadays, but it still takes an engineer to get his data for him, and to use his strains after he has them, and is likely to.

GEO. F. SWAIN, Assoc. Am. Soc. C. E.—The able and interesting paper by Mr. Wilson brings up a number of points which will bear discussion, but I will only touch upon one or two.

In these specifications we find for the first time, to the knowledge of the writer, the formulas of Launhardt and Weyrauch adopted as a basis for the computation of dimensions. But although this may be the first time that any of the newer formulas have been specified, our bridge-builders have not neglected to profit by the experiments of Wöhler, and to adapt their practice to his results. These experiments, and the extended discussions of the past ten or fifteen years, have principally led to a more general recognition and a clearer perception of the principle that the greater the difference between the maximum and minimum loads on a bar, the less should the maximum be. Formerly the so-called factor of safety covered everything beyond the mere statical stresses, including inaccuracies in calculation, effect of repetition of loads, impact, vibration, unequal distribution, possible flaws in the material, deterioration, etc. These experiments have enabled us to take account mathematically of the effect of repetition of loads, but they still leave every engineer to use his own judgment in taking account of the other unknown factors. In allowing for impact and vibration, and inequalities in the distribution of the live load, the writer of the specifications has, it seems to me, not proceeded logically. These factors, depending as they do upon the live load alone, can only be logically allowed for by adding percentages to the static stresses produced by that live load, or to the live loads themselves, as is already so frequently done. If the sudden changes in these percentages be objected to, a sliding scale or a

formula for different spans (as of floor-stringers) may be adopted. If the original form of Launhardt's formula, taking account of repeated applications alone, and not of vibration or impact, be assumed as

$$a = u\left(1 + \frac{\text{min. } B}{2 \text{ max. } B}\right),$$

then the correct way to allow for impact, etc., would seem to be to increase maximum B; not to multiply it by a constant, however, but to increase by a certain percentage that part which is due to the live load. Instead of this, exactly the opposite course is pursued in these specifications; maximum B is diminished by one-half, and then, to make up for this, u, which should have nothing to do with the question, is diminished. Regarding the actual value assumed for u in the formula adopted, it seems to the writer that it might reasonably have been made larger, and that a bar of good wrought-iron could be safely strained to over 15 000 pounds per square inch if the load were perfectly quiescent.

The writer's experience has been that the most convenient method of taking account of Wöhler's experiments, for all cases, is the one given by Winkler, which is really in principle nothing more than that first made use of in practice by Gerber, but recommended by John Griffen and Thomas C. Clarke, Ms. Am. Soc. C. E., in their paper before this Society many years ago, on "Loads and Strains of Bridges." As I do not remember to have seen these formulas quoted in American literature, I may be pardoned for referring to them here. They may be of some interest and novelty to some Members of the Society.

Winkler derives from Wöhler's experiments the following equations, which take account of repeated applications, but not of impact, etc.

Let f = area of cross-section; P max. and P min., the maximum and minimum stresses respectively. Then Winkler finds

$$\text{for wrought-iron} \begin{cases} \text{in tension principally: } f = \dfrac{P\text{ max.} - 0.45\, P\text{ min.}}{0.55\, K}; \\ \text{in compr'n principally: } f = \dfrac{P\text{ max.} - 0.40\, P\text{ min.}}{0.60\, K_1}; \end{cases}$$

$$\text{for steel} \begin{cases} \text{in tension principally: } f = \dfrac{P\text{ max.} - 0.56\, P\text{ min.}}{0.44\, K}; \\ \text{in compr'n principally: } f = \dfrac{P\text{ max.} - 0.63\, P\text{ min.}}{0.37\, K_1}; \end{cases}$$

in which K and K_1 are the allowable stresses per square inch for tension and compression respectively, for a purely quiescent load. These may be assumed according to the judgment of the engineer and the

material to be used, but when once chosen, the formulas become very simple. Winkler takes for wrought-iron $K = 20\ 000$, and $K_1 = 17\ 000$; for steel, $K = 25\ 500$, and $K_1 = 31\ 000$. Perhaps we should better accord with American practice by assuming for wrought-iron $K = 18\ 000$, $K_1 = 16\ 000$. We should then have, in round numbers,

$$\text{for tension principally: } f = \frac{P \text{ max.}}{10\ 000} - \frac{P \text{ min.}}{22\ 000};$$

$$\text{for compr'n principally: } f = \frac{P \text{ max.}}{10\ 000} - \frac{P \text{ min.}}{24\ 000}.$$

Suppose, in the case of tension principally, that the dead load causes a tension P_0, and the live load a tension P_1 and a compression P_2; then P max. $= P_0 + P_1$, and P min. $= P_0 - P_2$. Hence the first formula reduces to

$$(\text{tension}): f = \frac{P_0}{18\ 000} + \frac{P_1}{10\ 000} + \frac{P_2}{22\ 000} = \frac{P_0 + 1.8\, P_1 + 0.82\, P_2}{18\ 000}.$$

In most cases P_2 will be zero, and we shall have

$$f = \frac{P_0 + 1.8\ P_1}{18\ 000}.$$

This method simply amounts, then, to multiplying the live stress by a factor to reduce it to dead, and using 18 000 pounds per square inch as the allowable stress for a quiescent dead load. Winkler allows for impact by multiplying P_1 by a further factor, but he uses simply two factors, one for street and the other for railroad bridges. He would therefore presumably use the same factors for the floor-beam hangers as for the upper chords. This is, of course, incorrect, but the writer believes that by using a scale of percentages to allow for impact, etc., and then applying a formula like the one just given, the demands of modern practice may be met in the simplest and most scientific manner.

The clause in the specification explaining the mode of calculating the upper chord when the load rests upon it between the joints, appears rather complicated, and it seems to the writer that it would be just as well, in cases where this unfavorable arrangement is necessary, to proportion the chord for the direct thrust and the bending, considering the chord-piece as a supported beam simply; especially in view of the uncertainty attending the ordinary method of calculating pieces for compression and flexure.

These specifications require that the flanges of plate girders shall be calculated to bear the entire moment, and the web the entire shear. The writer is aware that this is the general practice, but he has never been

able to see the slightest reason, in this case, for taking things other than they actually are. The fact is, that the web does bear part of the moment; and if the factor of safety to be used, the allowance for impact, the effect of repetition, etc., be rationally taken into account, why should any further allowances be made? The writer sees, every day, girders in which the moment borne by the web has probably been neglected, and in which the web-splice has evidently been also proportioned to bear the shear alone, thereby no doubt putting an excessive strain on the rivets in the splice, and making a girder which is not of equal strength throughout. As the web really does bear part of the moment, it seems only reasonable that the splice should be calculated for it, and why not the rest of the girder also?

MACE MOULTON, M. Am. Soc. C. E.—Some two years ago I had occasion to use a modification of Launhardt's formula in proportioning the members of several spans of varying lengths.

The differences in the allowable unit strains from those I had previously used were so great, that I concluded to investigate, in a general way, the working values as given by Launhardt's formula applicable to a large range of spans, and also to discuss the formula in connection with the records of the experiments on which it was founded.

The results, as to unit strains and factors of safety which were obtained, are shown in the table on page 450.

The comparison was for simplicity confined to tension members in bottom chord, and for compression to the end posts. This was easily made by assuming that the strains in center bottom chord and in end posts were directly proportional to the total loads on span when fully loaded. Hence for any span:

$$\frac{\text{Min. } B.}{\text{Max. } B.} = \frac{\text{Total dead load.}}{\text{Total dead} + \text{total live.}}$$

The engines and train assumed were same as specified by Mr. Wilson, equated for each span. The weights of spans were from data at hand, partly actual and partly calculated for the purpose, closely enough for purposes of comparison.

The results of Launhardt's formula in tension are shown in column 4, and in compression in column 12. To tabulate results of common practice, with allowances for impact and influence of increased percentage of live load as compared with dead, columns 1 and 2 for tension, and 10 for compression were formed.

Length of Span.	For Tension. 10 000 pounds per Square Inch, with Impact Deducted.			For Tension. Launhardt's Formula: $a = u\left(1 + \frac{\text{min. } B}{2 \text{ max. } B}\right)$ $u = \frac{30\,000}{3} = 10\,000$		For Tension. Factor 4 for Dead. Factor 6 for Live. $a = 12\,000\left(-4\,000 + \frac{\text{live}}{\text{live+dead}}\right)$		For Tension. Wilson's Formula: $a = u\left(1 + \frac{\text{min. } B}{\text{max. } B}\right)$ $u = 7\,000$ for Shapes. $u = 7\,500$ for Double Refined Iron.		For Comp. 8 000 pounds per Square Inch, with Impact Deducted.		For Comp. Launhardt's Formula: $a = u\left(1 + \frac{\text{min. } B}{2 \text{ max. } B}\right)$ $u = 10\,000$ pounds.		For Comp. Factor 4 for Dead. Factor 6 for Live. $a = 10\,000\left(-3\,300 + \frac{\text{live}}{\text{live+dead}}\right)$		For Comp. Wilson's Formula: $a = u\left(1 + \frac{\text{min. } B}{\text{max. } B}\right)$ $u = 6\,500$ pounds.	
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Feet.	Impct pr. ct.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.	Unit Strain.	Factr.
20	35	7 600	6.	10 500	4.6	8 400	5.7	7 700	6.	6 080	6.6	10 500	3.8	7 030	5.7	7 150	5.6
30	30	7 940	5.8	10 650	4.5	8 520	5.6	7 910	5.8	6 350	6.3	10 650	3.7	7 129	5.6	7 350	5.4
40	25	8 200	5.6	10 800	4.4	8 640	5.6	8 120	5.7	6 540	6.1	10 800	3.7	7 230	5.5	7 540	5.3
50	22.5	8 400	5.5	10 900	4.4	8 720	5.5	8 260	5.6	6 720	6.	10 900	3.7	7 290	5.5	7 670	5.2
60	20	8 600	5.4	11 000	4.4	8 800	5.5	8 400	5.5	6 890	5.8	11 000	3.6	7 360	5.4	7 800	5.1
70	17.5	8 800	5.2	11 100	4.3	8 880	5.4	8 540	5.4	7 040	5.7	11 100	3.6	7 430	5.4	7 930	5.
80	15	9 000	5.6	11 200	4.3	8 920	5.4	9 320	5.4	7 170	5.6	11 200	3.6	7 460	5.4	8 060	5.
100	10	9 300	5.4	11 300	4.3	9 080	5.3	9 450	5.3	7 450	5.4	11 300	3.5	7 590	5.3	8 190	4.9
130	4	9 750	5.1	11 400	4.2	9 120	5.3	9 600	5.2	7 780	5.1	11 400	3.5	7 620	5.2	8 320	4.8
150	0	10 000	5.	11 500	4.2	9 200	5.2	9 750	5.1	8 000	5.	11 500	3.5	7 690	5.2	8 450	4.7
175	0	10 000	5.	11 700	4.1	9 320	5.2	10 050	5.	8 000	5.	11 700	3.4	7 790	5.1	8 710	4.6
200	0	10 000	5.	11 850	4.1	9 480	5.1	10 275	4.9	8 000	5.	11 850	3.4	7 920	5.	8 905	4.5
250	0	10 000	5.	12 250	3.9	9 760	4.9	10 875	4.6	8 000	5.	12 250	3.3	8 150	4.9	9 425	4.2
300	0	10 000	5.	12 350	3.9	10 080	4.8	11 025	4.5	8 000	5.	12 350	3.2	8 420	4.7	9 555	4.2
350	0	10 000	5.	12 550	3.8	10 320	4.7	11 325	4.4	8 000	5.	12 550	3.2	8 610	4.6	9 815	4.1
400	0	10 000	5.	13 100	3.7	10 480	4.6	12 150	4.1	8 000	5.	13 100	3.1	8 750	4.6	10 530	3.8
450	0	10 000	5.	13 400	3.6	10 720	4.5	12 600	4.	8 000	5.	13 400	3.	8 940	4.5	10 920	3.7
500	0	10 000	5.	13 500	3.6	10 840	4.4	12 750	3.9	8 000	5.	13 500	3.	9 040	4.4	11 050	3.6

The unit strains there shown are the equivalent strains, starting with a basis of 10 000 pounds per square inch in tension and 8 000 pounds in compression, which would give the same sectional area as if the impact percentages were added to the live load.

I then attempted to formulate the common practice ignoring impact, combined with a recognition of Wöhler's law, using the factors of 4 for all dead load and 6 for all live load.

The results of this are shown in columns 6 and 14, where general ultimates of 48 000 and 40 000 pounds per square inch were assumed. Columns 5 and 7 were also formed on the basis of an ultimate strength of 48 000 pounds and Wöhler's law, that rupture of material may be caused by stresses from repeated applications of loads, none of which equal the breaking load. The differences of these stresses being measures of the strength of the material, has, I believe, been fully demonstrated by his own and Spangenberg's experiments on small pieces, as well as those by Fairbairn on riveted girders.

These experiments showed that near the elastic limit a certain variable load was one and a half to twice as severe in its effect as the same load when quiescent.

From this it became apparent that engineers should take into account the effect of moving loads on a structure as distinguished from dead loads.

That this has been done, and that it enters to a certain degree into the every-day practice of American engineers, is evinced by the quotations in Mr. Wilson's paper.

Since reading his paper I have continued the comparison to include results from proposed formula. These are shown in columns 8 and 16.

I have also determined the factors of safety shown in columns 3, 9, 11, 17 referred for tension to ultimate strength specified, viz.: 46 000 pounds per square inch for plates and shapes (used up to 80 feet span), and 50 000 pounds for eye-bar material. All factors for compression are referred to 40 000 pounds per square inch ultimate strength.

The question which arises in the examination of the foregoing table, is whether the practice as generally pursued carries the principle of Wöhler through consistently, and whether the formulas and constants given by Mr. Wilson give more rational results than those heretofore used.

Launhardt's formula, using his divisor of 3 for working strength,

averages about a factor of 3 for dead and 5 for live loads, while that used in specifications amounts to using factor of 3 for dead and 6 for all live. The present general practice more nearly approaches factors of 4 and 6 for dead and live respectively, all referred to the ultimate strength of the material.

If consistently carried out, the latter would be as before stated, about as shown in columns 6, 7 and 14, 15. If no allowance for impact, or the principle enunciated by Wöhler was made, the uniform factor would, of course, be 5.

Study of the results of experiments before mentioned did not to me seem to wholly justify the use of Launhardt's formulas as his and Weyrauch's constants left it, neither did the common practice seem to sufficiently comply with the law of variations in effect of live and dead loads. Nevertheless the latter was certainly safe within limits discussed.

The formula proposed, however, seems to me to fur ish a close approximation for tensile stress to the conditions impo. . by the loads. It assumes that the effect of live load is twice as injurious as that of dead load.

This was the extreme limit of variation found by experiment near the elastic limit of the material, and therefore we have a corresponding and additional margin of safety for strains never exceeding one-half the elastic limit.

For compressive strains I should consider it equally good for upper flanges of well-braced plate or lattice girders under 80 feet span.

In its application to the constants of the well known column formulas, we have, however, a different case to consider.

These formulas, as adopted by Mr. Wilson, with working strain of 8 000 pounds per square inch, decreased in proportion of length to least radius of gyration, give results agreeing as closely as any with those of testing machine on square columns.

The application of the loads applied in the testing machine being gradual, the post may be assumed to be in a condition similar to that in the structure with a gradual livel oad coming on, without shock, of a sufficient intensity to cause rupture.

Taking the specified formula for permissible working stress for all live loads we have $a = 6\ 500$ pounds per square inch, which corresponds as to conditions with the 8 000 pounds in column formulas, and which gives a factor of 6 as referred to 40 000 pounds ultimate strength per square inch in compression.

It has been proven by a multitude of experiments, that when no lateral flexure occurs, the rupture of any structural material, subjected to bending stress, occurs on the side which is strained in tension. From this fact, it is fair to assume that when the column is properly stayed to prevent flexure from loads producing stresses in the material up to the elastic limit, the material composing the column is as well able to take compressive as tensile stress.

Of course, in practice, our columns are designed to admit working stresses only which are far within the elastic limit. Hence, it seems rational to allow a working value equal to that for tension in the same material.

Mr. Wilson's constant is, for compression, less than that for tension, hence, still farther on the safe side.

I will state that the percentages added to live load in table, under head of impact, were obtained by using data afforded by Mr. B. Baker, which he deduced from experiments by Baron Von Weber, and applied on the heaviest English locomotives. Mr. Baker allowed a percentage for impact for short spans to be added to the live load, and also decreased his unit strain.

Percentages given simply combine these operations, and are applicable to any other live loads, as well as those considered by Mr. Baker.

A. P. Boller, M. Am. Soc. C. E.—In commenting upon Mr. Wilson's paper, I recognize the fact of his unusual opportunities for noting the behavior of iron in structures under the wear and tear of actual service; opportunities denied the regular professional bridge-builder, who rarely sees his work after it is once taken off his hands, unless he makes a special mission for the purpose. The first point brought out by Mr. Wilson is the purely arbitrary practice among engineers in their selection of what may be termed the base strain for proportioning structural work, and his recommendation of the Launhardt and Weyrauch formula is in the direction of uniformity of practice. No accurate formula can ever be devised, any such being as arbitrary as methods in common use; but this one of Launhardt's seems to crystallize in a simple way the recognized truths, that the greater the ratio between dead and moving load, the less should material be strained; and that where material is subject to alternate tension and compression, it should be proportioned with a unit strain largely in excess of what would be required

for all tension or all compression. The formula is certainly on the side of safety, having been worked out from deductions based upon impact experiments, involving conditions to which few, if any, bridge structures are actually subjected. Impacts continuously kept up to rupture, on comparatively small pieces, create a ceaseless molecular vibration that even our elevated railways are not subjected to; and it seems to me going to the extreme of conservatism to judge of the value of iron construction wholly on the same basis. It is impossible for the engineer to be bound entirely by an empirical formula, no matter how convenient its use, as varying circumstances will control his judgment as to permissible strains. A bridge structure infrequently used, or one which, in the nature of things, can never be subjected to a high-speed traffic, may safely be proportioned for a higher unit strain than structures subjected to great speed or in frequent use. Why Mr. Wilson should assign a less compressive than tensive value to wrought-iron is not clear, and this is hardly in accord with modern experimental knowledge. The elastic limit in compression is fully as high as in tension, and the ultimate compressive strength, where bending is not in question, is at least equal to the highest tension value, and in large sections certainly higher. Then why make the distinction?

I question very much Mr. Wilson's assumption that continuous compression members over points of support are substantially hinged columns, and should be so treated. This is an important fact, if true, as it would add a large percentage of metal to upper chord members as usually proportioned. It seems to me that such a chord is held as securely by the pins passing through it as if riveted by gusset plates, or supported direct on post heads, and that the assumption there is a tendency for compound bending is an over-refinement that has very serious tendencies, not only adding quite perceptibly to the amount of material in a given structure, but also on the practical questions involved in long spans. There is one point in the specifications that will, I think, strike many engineers as exceedingly critical of common practice, and that is the condemnation of plate girders having no upper flange plates. I am sure most engineers will agree that any exact theoretical solution of the plate-girder problem is impossible, and that the fewer the parts of which such a girder is composed, the more nearly can theory and practice be made to harmonize. Consequently, so far from not insisting upon plating the flanges, it is desirable to retain but a pair of

angles for each flange, as far as practicable, even at the expense of more metal, and only have recourse to plating when constructive necessity demands it; and even in that case not to build up theoretical sections with too many thin plates. Piling up thin plates for the sake of economy of metal can only be done at the sacrifice of the economy of the shop. I know of no branch of constructive iron-work that demands more sound, common sense than plate-girder work, in which simplicity and fewness of integral parts are base principles.

MANSFIELD MERRIMAN, M. Am. Soc. C. E.—The method of assigning the working stresses per square inch for members subject to alternating stresses, by means of the formulas of Launhardt and Weyrauch, appears more rational and satisfactory than the use of percentages varying with length of span or with the ratios of dead to live load. It is, however, an objection to Weyrauch's formula that it does not contain t, the ultimate strength of the material under a static stress. Launhardt's formula contains t, but the vibration strength s is absent, and this would probably affect the value of the working stress a for a small range of stress near the zero point.

Let R denote the ratio of the least limiting stress in the member to the greatest limiting stress. If these limiting stresses are both tension or both compression, R will be positive, but if one is tension and the other compression, R is negative. Then the formulas of Launhardt and Weyrauch may be written

$$a = u\left(1 + \frac{t-u}{u} R\right) \text{ for positive } R.$$

$$a = u\left(1 + \frac{u-s}{u} R\right) \text{ for negative } R.$$

Here there are two formulas to express the variation of a as R varies from $+1$ to -1, whereas it would seem that one formula expressing a as a continuous function of R in terms of t, u, and s would be a more rational expression of the true law.

The following formula, whose deduction is elsewhere given, contains u, t, and s, and gives values of a corresponding to values of R from $+1$ to -1:

$$a = u\left(1 + \frac{t-s}{2u} R + \frac{t+s-2u}{2u} R^2\right) \text{ ....(1)}$$

This formula satisfies the three limiting conditions; thus, if $R = +1$,

the value of a is t; if $R=0$, the value of a is u; and if $R=-1$, the value of a is s; and these are the three values indicated by the experiments for a great number of repetitions of stress between the limiting values.

For wrought-iron, experiments show that $t=2u$ and $s=\frac{1}{2}u$. Hence formula (1) becomes

$$a=u\left(1+\frac{3}{4}R+\frac{1}{4}R^2\right)\text{..............................}(2)$$

To use this formula for determining values of the working unit stress a, the value of u must be taken as one-half of the working strength per square inch for a static load liable to no variation. For example: If the working strength under a static stress be taken as 14 000 pounds per square inch, the formula is

$$a=7\,000\left(1+\frac{3}{4}R+\frac{1}{4}R^2\right)$$

Now for a member whose stress ranges from 40 000 pounds tension to 100 000 pounds tension, the value of R is $+\frac{4}{10}$ and that of a is 9 380 pounds per square inch; but for a member whose stress ranges from 40 000 pounds compression to 100 000 pounds tension, the value of R is $-\frac{4}{10}$ and the value of a becomes 5 180 pounds per square inch.

On Plate XL are plotted the curves corresponding to formula (2) for the three cases, $u=7\,500$, $u=7\,000$, and $u=6\,500$ pounds per square inch. The straight lines corresponding to the formulas of Launhardt and Weyrauch are also drawn for the case of $u=7\,500$. The diagram shows that the formula (1) agrees with those of Launhardt and Weyrauch when $R=+1$, $R=0$, and $R=-1$; but that for all other values of R, it gives somewhat smaller values of the allowable working unit stress a. Launhardt's formula is that of a straight line, Weyrauch's formula is also a straight line intersecting the first, while (1) is the equation of a parabola passing through the three points best determined by experiments.

The very few experiments made by Wöhler for values of R between $+1$ and 0 seem to be somewhat better satisfied by Launhardt's formula than by formula (1). No experiments were made for values of R between 0 and -1.

The following table gives values of a corresponding to the three curves shown in the diagram:

	$(u=7\,500)$	$(u=7\,000)$	$(u=6\,500)$
R	a	a	a
+1.0	15 000	14 000	13 000
+0.9	14 081	13 143	12 204
+0.8	13 200	12 320	11 440
+0.7	12 356	11 533	10 709
+0.6	11 550	10 780	10 010
+0.5	10 781	10 063	9 344
+0.4	10 050	9 380	8 710
+0.3	9 356	8 733	8 109
+0.2	8 700	8 120	7 540
+0.1	8 081	7 543	7 004
0.0	7 500	7 000	6 500
−0.1	6 956	6 493	6 029
−0.2	6 450	6 020	5 590
−0.3	5 981	5 582	5 184
−0.4	5 550	5 180	4 810
−0.5	5 156	4 813	4 469
−0.6	4 800	4 480	4 160
−0.7	4 481	4 183	3 884
−0.8	4 200	3 920	3 640
−0.9	3 956	3 693	3 429
−1.0	3 750	3 500	3 250

The value of a may be expressed also in the following manner:

$$a=p\,S. \qquad \text{(3)}$$

in which S is the working strength $2\,u$ for a purely static stress, and p is a number less than unity or the percentage to be taken of S for a particular value of R. By comparison with formula (2) it is seen that the value of p is

$$p=\frac{1}{2}\left(1+\frac{3}{4}R+R^2\right)$$

The following table gives values of the percentage p for different values of R. When R has been found for any particular case p may be taken from this table, and then the permissible working unit-stress a is p times the greatest allowable working unit-stress S.

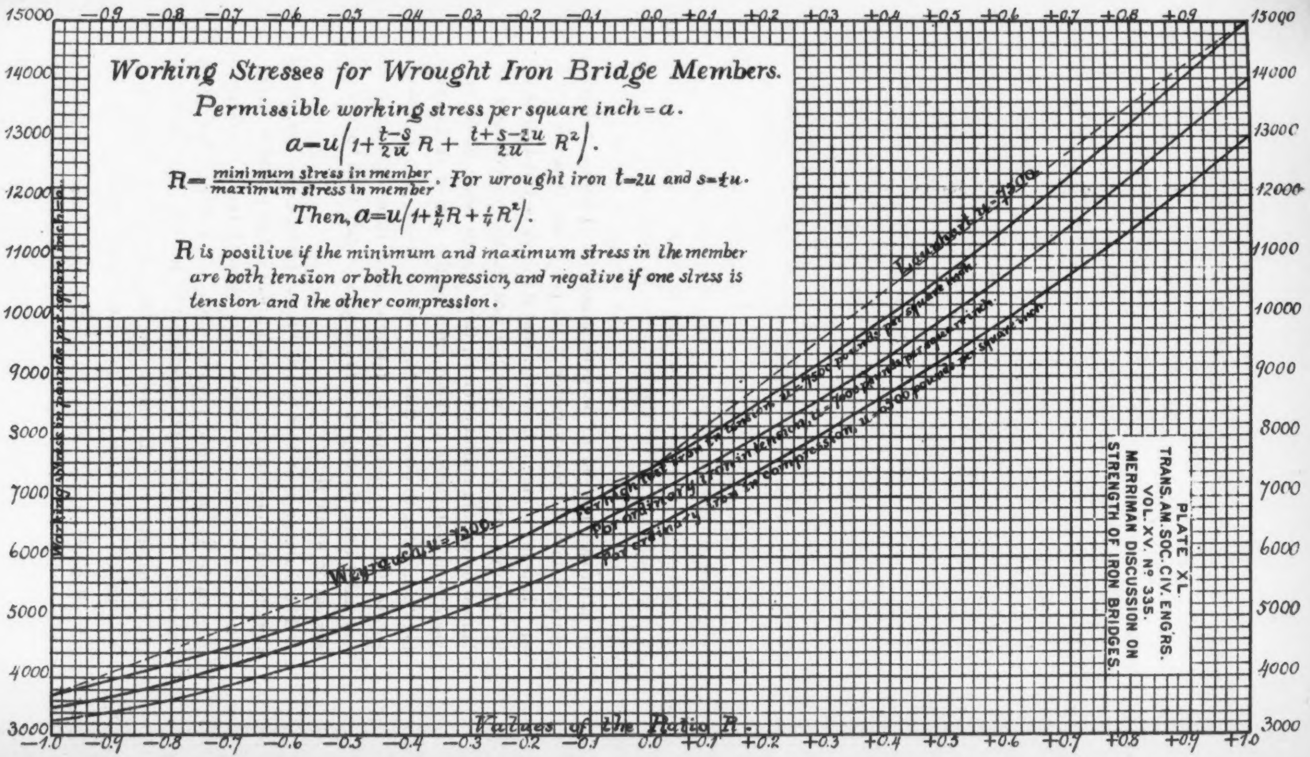
R	p	R	p
+1.0	1.000	+0.4	0.67
+0.9	0.939	+0.3	0.624
+0.8	0.88	+0.2	0.58
+0.7	0.824	+0.1	0.539
+0.6	0.77	0.0	0.50
+0.5	0.719		

R	p	R	p
0.0	0.50	—0.6	0.32
—0.1	0.464	—0.7	0.299
—0.2	0.43	—0.8	0.28
—0.3	0.399	—0.9	0.264
—0.4	0.37	—1.0	0.25
—0.5	0.344		

James G. Dagron, C. E.—The author, in his specification requires that the eyes on all tensile members shall have fifty (50) per cent. excess of material at the pin when the diameter of pin does not exceed the width of bar, and one hundred (100) per cent. excess when the diameter is twice the width of bar or over; for intermediate sizes of pins the excess of eye to be made proportional to their diameter. An additional clause specifies that all links and rods, if tested to destruction, shall part through the body and not through the head or pin-hole.

These requirements do not agree with the results of many tests made by myself, during the past year, on eye-bars manufactured by three of the leading bridge companies; in fact these experiments have shown that the best results are obtained when the diameter of the pin is equal to or greater than the width of the bar, provided proper care is taken in the manufacture of the head. The proper amount of excess to be given in the latter case has yet to be determined, but I think it should not be greater than when the diameter of the pin is less than the width of the bar. It is my opinion, however, that it would be injudicious to reduce the requirements of the specifications to the percentages of excess claimed by certain manufacturers, these claims being based upon a few exceptional tests such as those cited by the author in his paper, and there is nothing to prove that they can be obtained in a current manufacture, for it should not be forgotten that these special test-bars are very liable to be the object of much greater care than bars of a current fabrication would receive.

The percentage of bars which break through the head or pin-hole when tested to destruction, is much greater than would be supposed. It is true, however, that a certain proportion of these breakages may be chargeable to improper proportioning of width of bar for a given diameter of pin. Of the 125 bars tested by me, fifty-two (52) were of steel and the balance iron. The steel bars ranged in width from 3 to 7 inches, the smallest section being 3 by $\frac{13}{16}$ inches and largest 7 by $1\frac{1}{8}$ inches. The diameter of the pins varied from $4\frac{1}{8}$ to $6\frac{7}{8}$ inches, and in the


Working Stresses for Wrought Iron Bridge Members.
Permissible working stress per square inch = a.
$a = u\left(1 + \frac{t-s}{2u}R + \frac{t+s-2u}{2u}R^2\right).$
$R = \frac{\text{minimum stress in member}}{\text{maximum stress in member}}$. For wrought iron $t = 2u$ and $s = \frac{1}{2}u$.
Then, $a = u\left(1 + \frac{3}{4}R + \frac{1}{4}R^2\right).$
R is positive if the minimum and maximum stress in the member are both tension or both compression, and negative if one stress is tension and the other compression.
PLATE XL.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XV. Nº 335.
MERRIMAN DISCUSSION ON
STRENGTH OF IRON BRIDGES.
Launhardt $u = 7300$
Weyrauch $u = 7300$
For high test iron in tension $u = 7500$ pounds per square inch
For ordinary iron in tension, $u = 7000$ pounds per square inch.
For ordinary iron in compression $u = 6300$ pounds per square inch
Working stress in pounds per square inch $= a$.
Values of the Ratio R.
−1.0
−0.9
−0.8
−0.7
−0.6
−0.5
−0.4
−0.3
−0.2
−0.1
0.0
+0.1
+0.2
+0.3
+0.4
+0.5
+0.6
+0.7
+0.8
+0.9
+1.0
3000
4000
5000
6000
7000
8000
9000
10000
11000
12000
13000
14000
15000

majority of cases was greater than the width of bar. The iron bars broken were mostly five (5) and six (6) inches in width, the pins in the majority of cases being of a diameter less than the width of bar. The percentages of excess of material through the pin-hole varied from fifty (50) to sixty (60) per cent. for both steel and iron bars. The majority of the steel bars were made by the hydraulic upsetting and forging process, and notwithstanding the care taken in their manufacture, nearly twenty-nine (29) per cent. of the total number tested broke through the pin-hole or in the head, and of these over fifty-seven (57) per cent. had pins of a greater diameter than the width of the bar; the excess of material through the pin-hole in every case being about fifty (50) per cent.

The iron bars were mostly made by upsetting and piling, and in this case nearly forty-four (44) per cent. broke through the pin-hole or in the head. The excess of material through the pin-hole was in every case greater than fifty (50) per cent.

The bars broken were not special test bars made for the occasion, but were taken from the piles of bars intended for use in the different structures, and can thus be fairly supposed to represent the present stage of eye-bar manufacture.

The results obtained have shown that there is considerable room for improvement in this line, and that until it is conclusively proven that this improvement has taken place it would be unwise to reduce the percentages of excess, especially in the case of steel bars, where so many additional factors, such as proper heating and annealing, tend to complicate the problem of successful manufacture.

William Sellers, M. Am. Soc. C. E.—The paper concludes with the statement: "Great care has been taken in framing the conditions for quality of material and workmanship to insure having what is desired, and what the present advanced state of manufacture can produce, without asking for what is impossible to obtain except at extraordinary cost." I propose to confine my discussion of this paper to that portion of the specification thus referred to, believing that, in the present state of the art, less difference of opinion exists as to the strains and the methods of computing them, than as to the details of construction and the quality of material and workmanship. And at the outset I have to say that the author is evidently sincere in his desire not to burden his specifications

with requirements that cannot be complied with, except at an extraordinary cost, and in the main he has been successful. But there may be requirements, the fulfillment of which cannot be determined by the physical tests prescribed; that nothing, in short, but the personal supervision of the engineer, or his inspector, at every stage of the manufacture could enforce, and as this last is practically impossible, he finally contents himself with the behavior of his material under his physical tests. Now the question arises: Is it wise to retain provisions that never are and never can be enforced? I allude to the clause "must be double-rolled after and directly from the muck bar (no scrap will be allowed)." To comply with this provision literally would certainly involve additional cost, whereas, with rare exceptions, no extra charge is made by the manufacturer when this provision, common to most specifications, is brought to his notice. There are exceptions, who say they will agree to comply with the physical tests, but they cannot conscientiously say they will comply with this provision of manufacture. I believe it is as unwise as it is unfair to prescribe conditions for manufacturing which practically prevent such men from furnishing this class of material, and it would be well for the engineer not only to limit his requirements to what is desired, but also to determine what means are at his disposal for insuring that he will obtain what he desires, and then limit his requirements strictly to his means for insuring their fulfillment. In framing specifications it must be remembered that they are to be construed by inspectors who are generally innocent of any practical knowledge of the material with which they are dealing, or of modes of construction, or of the means for its accomplishment; and too much care cannot be taken to avoid ambiguities and the exercise of their judgment. And to this end I suggest that the words "hereinafter described" should be inserted after test piece in the first, and "in addition to the first requirements" should be appended to the second, third and fourth paragraphs of Quality of Materials.

The test piece as described contemplates a uniform length with varying thicknesses. It is desirable that the ductility should be comparable in all test pieces; and this cannot be done unless the length is a uniform multiple of their shortest transverse section. It is impossible to elucidate this proposition within the limits of this discussion, but I may refer to a paper by W. Hackney, read before the Society of Civil Engineers in London, and published in Vol. 76, Session 1883-84, Part II, in which the advantages of this modification are clearly set forth.

Under Workmanship an omission occurs which I believe has escaped the notice of the author. It should read, "In the case where rough bolts are permitted a variation of one-sixteenth of an inch will be allowed between diameter of bolt and hole."

Since the report of tests on steel eye-bars was made which the author has quoted, the Edge Moor Iron Company have nearly completed the modification of their plant for the manufacture of such bars, but I am unable as yet to submit any later data than that contained in the paper. It is to be regretted that the author has limited his specifications to the use of wrought-iron only, for at the present day steel enters largely into the composition of nearly all bridges, and his formulas are as well adapted to the one material as to the other. So that it only requires that the requisite tests should be prescribed to obtain the quality with reasonable exactitude, without resorting to tests that add materially to the cost, while they add literally nothing to the security of the structure, and I suggest that the discussion might be profitably extended to this branch of the subject.

E. Thacher, M. Am. Soc. C. E.—Although I do not fully agree with the author in all particulars, I think his specification is a move in the right direction, and that it will give safe and fairly economical results. From the standpoint of a bridge engineer it is too complex; too much time is required in estimating; and essentially the same results can be obtained by simpler means. The specification gives three different distributions of rolling load, the maximum of these being used in proportioning every member of the structure. Now, for any length of span or class of engine, a uniformly distributed load can be found, which, when used in calculation, will give essentially the same results as the wheel loads; and by aid of preliminary moment tables these maximum equivalent loads can be readily calculated and tabulated once for all, and the various competitors for work would be relieved from the necessity of devoting hours of time to calculations which could be made as well in a few minutes; and as the loads are typical at best, great exactness in this regard does not appear to me essential. Having fixed upon the loads, I would have the stresses correct for those loads, which is not the case if the cross-girder load under the drivers is considered the head of the train, as the author recommends. To be sure it is on the side of safety, but not consistently so, as the percentage of gain varies largely in the different members.

The division of dead load at panel points, as specified, requires too much labor, and it cannot well be observed except by revision. My practice has been to consider two-thirds panel weight at loaded and one-third at unloaded apex, which is believed to be sufficiently correct for all practical purposes.

I have confidence in the correctness of Launhardt and Weyrauch's formulas. The ordinary arbitrary provisions for repetitions of load and impact are inconsistent and unsatisfactory. The modification of Launhardt's formula adopted by the author, allows $33\frac{1}{3}$ per cent. as the maximum effect of impact alone, which appears to me about right. Ordinarily the track on bridges is the best on the road, and under these circumstances the effect of impact from various recorded experiments appears to be next to nothing; but to provide for imperfections and to a certain extent, accidents, it is well to make a liberal allowance.

The formulas of Launhardt and Weyrauch allow for the fatigue of the material, but I can see no good reason why their results should be used in Rankine's formulas for struts which fail partially or wholly by flexure; nor, in fact, much necessity for Rankine's formulas. Iron struts deflect very little until near the breaking point, and repetitions of stress within safe or working limits are not repetitions of flexure. If Rankine's formulas are used, I believe the numerator should be constant or nearly so, and that the resulting working stress should not exceed the allowed stress by Launhardt's formula.

For members subject both to tension and compression, I can see no relation between the tension to which it is subjected and its failure by flexure. I believe flexure formulas should consider compression only, but that the resulting working stress should not exceed the amount given by Weyrauch's formula for alternate stress.

Plates XXXV and XXXVI, comparing the results of experiments with various proposed formulas, and Plate XXXVII, comparing proposed formulas with each other, are very interesting and instructive. So long, however, as there remains such a wide discrepancy in opinion as to the proper value of the constants that should be used in Rankine's formulas, as illustrated by Plate XXXVII, resulting in a variation of from 60 to upwards of 100 per cent., it would appear to me that the formulas are of very little service. If on Plate XXXV we draw a line connecting the maximum values of a, and another connecting the minimum values, leaving out of consideration welded

tubes and swelled octagon columns, there will be inclosed an area showing the variations in the results of experiments on flat-ended struts. If now a line is drawn, starting at 45 000 at $\frac{l}{r} = 0$, and ending at 9 000 at $\frac{l}{r} = 240$, it will pass very nearly through the center of this area, and will apparently represent more nearly than any of the proposed curved lines, the average strength of the struts. The equation of this line is $a = 45\,000 - 150 \frac{l}{r}$ or allowing a factor of safety of 5, $a = 9\,000 - 30 \frac{l}{r}$ which is a simple formula, easily applied. Likewise, if on Plate XXXVI we draw lines connecting the maximum and also the minimum values of a, leaving out of consideration a few scattering results, due probably to exceptional character of material or manner of conducting the experiments, there will be inclosed an area showing the variations in the results of experiments on pin-ended struts. If now a line is drawn, starting at 45 000 at $\frac{l}{r} = 0$, and ending at 10 000 at $\frac{l}{r} = 175$, it will pass nearly through the center of this area, and will represent quite as nearly as any of the proposed curved lines the average strength of the struts. The equation of this line is $a = 45\,000 - 200 \frac{l}{r}$ or, allowing a factor of safety of 5, $a = 9\,000 - 40 \frac{l}{r}$ which is also a simple formula easily applied. To sum up, I would propose the following formulas for working stresses:

TENSION ONLY.

$$a = 7\,500 \left(1 + \frac{\text{min. stress in member}}{\text{max. stress in member}}\right) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

COMPRESSION ONLY.

$$\left.\begin{array}{l} b = \left(9\,000 - 30 \frac{l}{r}\right) \text{ flat ends} \\ b = \left(9\,000 - 35 \frac{l}{r}\right) \text{ one flat and one pin end} \\ b = \left(9\,000 - 40 \frac{l}{r}\right) \text{ pin ends} \end{array}\right\} \text{Not to exceed } a \ldots\ldots\ldots \quad (2)$$

ALTERNATE TENSION AND COMPRESSION.

For compression only, see formula (2).

For greater stress tension or compression

$$c = 7\,500 \left(1 - \frac{\text{max. stress of lesser kind}}{2 \text{ max. stress of greater kind}} \right)$$

use the one giving the greatest area of section.

I do not think it necessary that horizontal struts, continuous over points of support, should be considered as hinged at such points, as undoubtedly the weight of the piece is more than sufficient to overcome any tendency to bend upwards; but for members vertical, or nearly so, with intermediate points of support, such as posts and bridges in viaducts, I fully agree with the author.

I think it is very essential that the center of pin should always be placed as nearly as possible in the neutral axes of the section. Too little attention is usually paid to this. No provision can be made for taking up eccentric stresses except by large additions to the section. It is very common in the design of bridges to have the top chord and end braces consist of two rolled or riveted channels with equal flanges and a top plate, the center of pin being placed at the center of the channels. This plate is always counted as effective area, notwithstanding its addition is often a positive injury, the section being stronger without than with it.

I do not approve the method specified by the author for proportioning the top chords of deck bridges when the floor system rests directly on them. Such construction I consider poor, at best, and prefer not to use it if it can be avoided. The stresses are much affected by deflections at panel points, character of splices and position of pins, and cannot be determined with any degree of accuracy. If the chord is continuous, the author's method will give a weakness of bottom flange at panel point; and if not continuous, a weakness of both flanges at center of panel. I think the only way we can know that the material is not overstrained is, first, to place the center of pin at the center of gravity of the section, as the effect of eccentricity in a continuous chord is difficult to determine; second, to consider as a beam of one panel length, subject to the maximum bending that will result from wheel loads and floor system, the beam to be considered as supported at the ends for center section and fixed at the ends for end section. I would use the working

stress a in both cases, unless the direct thrust is in excess of the bending stress.

The rule given by the author for excess of material in the heads of eye-bars does not agree with the results of experiments at the Keystone Bridge Works. We have found beyond question that the larger the pin as compared with the width of bar, the less the excess required to develop the strength of bar. The author makes the allowed bearing between pins and pin-holes a function of his compressive unit stress a. This is very troublesome to apply, and I believe a uniform unit stress of, say, 12 500 pounds per square inch, will give fully as good results. He makes the maximum allowed fiber stress on pins vary with the maximum tensile stress a. I think this could remain constant without detriment to the work, and be simpler in application. W. S. Thompson, Manager of the Dominion Bridge Works, as the result of eight or ten careful experiments on small scale machine for fiber stress necessary to produce permanent set in iron and steel pins, found that set was produced in iron pins at a stress of from 45 000 to 47 000 pounds per square inch, and in steel pins at a stress of from 70 000 to 75 000 pounds per square inch; the length was taken in clear between supports, and the applied pressure was on a bearing $\frac{3}{8}$-inch wide at center. I have seen pins taken from old bridges after many years' service, and apparently uninjured, which, according to the ordinary way of figuring here, sustained a stress of 100 000 pounds per square inch and upwards. The usual requirement of considering all forces applied at bearing centers, frequently leads to very erroneous results, and certainly should not be followed in all cases. I am satisfied, however, that good sized pins are essential in equalizing the stresses on connecting members, and that an allowed maximum unit fiber stress of 15 000 pounds for iron, and, say, 21 000 pounds for steel, is good practice.

The author requires double nuts on upset rods. I do not think this desirable; a single nut locked by a dap with a narrow chisel at the point where the thread enters the nut, being cheaper, neater, and perfectly effective.

The author requires that rivets shall not be spaced further apart in line of stress than twelve times the thickness of the thinnest external plate connected. If the space meant is from center to center of rivets, I think sixteen times the thickness of plate is sufficient, and will give better work.

The author's rules for the use of lattice bars are simple and practical, and a vast improvement on the rules sometimes given, which to apply require more labor than all other calculations combined, and which, in my opinion, are utterly worthless in theory, and result in great waste of material. Lattice bars should be heavy enough to protect the member from injury in handling, but I am satisfied small sizes are sufficient after the member is once in position. Several years ago I witnessed an experiment at the works of the Louisville Bridge and Iron Company on a post consisting of two channels connected by light lattice bars. I have not a record of the test conveniently at hand; but, quoting from memory, the post began to fail about axis through center of web at a stress somewhat in excess of formula requirements, and recovered on removal of pressure. After repeating this three or four times, with about the same result, it was decided to try the effect of lattice bars. At first one or two were knocked off, and the pressure was applied without change in result; and this was repeated, removing additional bars at random after each trial, until wide and irregular gaps were left between flange supports, and at last the post was destroyed by buckling of flange. My rule has been to provide lattice bars as follows: $2\frac{1}{4}$ by $\frac{3}{8}$ for $\frac{7}{8}$-inch rivets; 2 by $\frac{5}{16}$ for $\frac{3}{4}$-inch rivets; $1\frac{3}{4}$ by $\frac{1}{4}$ for $\frac{5}{8}$-inch rivets; and $1\frac{1}{2}$ by $\frac{1}{4}$ for $\frac{1}{2}$-inch rivets; which is believed to be good practice.

The rule given by the author for the length of the tie-plates at the ends of compression members—using the ordinary bearing and shearing values of rivets—results in extremely long plates. These not only add quite materially to the cost of structure, but are troublesome to paint. I believe a good rule is to make the length of connecting plates equal to the depth of member, and I have no evidence that longer plates are of any advantage.

The author makes no allowance for the web in the calculation of the flange sections of plate girders. I think one-sixth of web should be counted as flange area in each flange. It is undoubtedly effective to this extent, and not to consider it is discriminating in favor of lattice girders, which, in my opinion, are much inferior. In such girders the connection of elastic web members to comparatively inelastic chords necessarily gives very unequal stresses on the rivets, of which there is usually a deficiency at best.

The author remarks that the gain from the consideration of web is

very small; but as it amounts to about $2.50 per lineal foot for his double-track bridges, it seems to me quite an important matter.

The author specifies that the flanges of plate girders exceeding 12 inches in width shall have at least 4 lines of rivets. I do not think it advisable to use more than 2 lines of rivets under any circumstances. There is usually no difficulty, however, in limiting the width to 12 inches. I think the compressed flange of beams and girders should be computed by Rankine's formula, as given by the author, except I would use a constant numerator, the resulting working stress not to exceed the amount given by Launhardt's formula.

To simplify calculation, and to allow for the use of tables, which are of the greatest assistance in design, I think the shearing and bearing values of rivets in plate girders should be constant, and that an allowance of 7 500 pounds per square inch for shearing, and 15 000 pounds per square inch for diametrical bearing, will give good results. As the rivets in all probability are rarely ever subjected to any shearing, or bearing either—the friction between the plates due to the contraction of the rivets in cooling, being, by various authorities, more than sufficient to prevent it—I do not think any great fineness of calculation necessary. It is not usually considered good practice to rely to any great extent upon friction, but it nevertheless exerts a very large force in favor of stability; and at least one very high authority thinks we are justified in making use of it if necessary.

The distribution of rivets as specified does not differ materially from ordinary practice, but does not appear to have been observed closely in the standard plans of the Pennsylvania Railroad; for instance, on standard plan of 75-foot girders the flange angles are connected to the web by 2 rows of $\frac{3}{4}$-inch rivets staggered, the rows having a pitch of 4 inches and 8 inches respectively. The flange plates and angles are connected by 4 rows of $\frac{3}{4}$-inch rivets, 2 rows having a pitch of 4 inches, the other 2 a pitch of 8 inches. On standard plan of 44-foot girders, a 12-inch top plate has 4 rows of $\frac{3}{4}$-inch rivets pitched 2 inches transversely. The number of rivets provided in these girders appears to be much in excess of specification requirements. Rivet holes are a source of weakness, and an excessive use of rivets not only adds largely to the cost of work, but in my opinion is a material injury to the girder.

I do not agree with the author on his theory of the stresses in the webs of plate girders, nor see how it can possibly be true unless the

web is disconnected between the lines of his principal stresses. I am satisfied that the theory of Professor Rankine and others is nearly correct. It is quite clear that in a girder in which the great mass of the material is concentrated in the flanges, the intensity of the shearing force is practically constant throughout the depth of the web, and that all the forces acting on any material particle can be resolved into two forces, one of tension and the other of compression, acting at angles of 45 degrees with the neutral axis. It is not so clear at first glance that the intensity of this shearing force, horizontally, vertically and diagonally $= \frac{F}{h}$ per foot run, in which F equals the shear and h the effective depth in feet; but the consideration of a lattice girder makes this quite clear. Suppose a lattice girder loaded uniformly, and having struts and ties one foot apart horizontally, and inclined at angles of 45 degrees with the neutral axis. Let N equal the number of systems of bracing, then $\frac{F}{N}$ equals the average shear on each brace; but if the braces are of uniform thickness, like the web of a girder, the width of each brace is equal to $\frac{1}{\sqrt{2}}$ and the shear per lineal foot of width of brace equals $\frac{F \times \sqrt{2}}{N}$ but as the braces are inclined at angles of 45 degrees, the stress on brace equals shear $\times \sqrt{2}$ equals $\frac{2F}{N}$ per foot run of width, and as the braces are 1 foot apart horizontally and vertically $N = 2h$, therefore the tension per foot run equals compression per foot run equals $\frac{F}{h}$ which agrees with Rankine's theory. The distortion of the web under a load is such as would be produced by stresses of both tension and compression acting at angles of 45 degrees, and with compressive forces conveniently at hand it does not appear reasonable that the tensile forces will go out of their way in search of stiffeners upon which to act. Examples are on record which show that girders sometimes give way by failure of the web. The following are taken from Professor Unwin's Treatise on Iron Bridges and Roofs.

In the sixteenth experiment preliminary to the construction of the Conway and Menai Tubular Bridges, the weakness of the web was manifest, the sides buckling very much near the ends, the ridge being inclined to an angle of 45 degrees, as theory would dictate. In the thir-

tieth experiment a tube failed in a similar manner. In the fiftieth experiment on a 66-foot girder, 10 feet deep, the web began to buckle with 110 tons, and collapsed with 165 tons. In the second experiment on the model of the Britannia Bridge, the weakness of the sides to resist thrust led to the failure of the girder; the sides being thrown into diagonal waves of puckering; it would seem therefore that the compressive stresses in the webs of girders are not imaginary.

Although I believe Rankine's theory to be about correct, I do not think it necessary to make so much provision to meet the compressive stresses as he recommends. Stoney shows that a very small force applied at right angles to the line of stress is sufficient to keep a long strut in line. The moment the compressive forces deflect the web, the opposite tensile forces exert a powerful influence to bring it back, these forces being applied at every point of its length. In addition to this, the flanges resist to a considerable extent any tendency to buckle in diagonal lines, so that altogether it is not surprising that a very thin web will withstand the compressive shearing stresses safely.

In the designs of plate girders I have usually considered the length of web strut equal to the clear depth between flange angles, which I believe to give ample provision against buckling, and the stiffeners required by this rule are within the limits of ordinary practice. Stiffeners at a distance apart much exceeding the depth of girder, I consider useless, except to assist in straightening the web if kinked.

Girders formed with web and angles alone are prohibited. Such girders for short spans, such as track stringers, I consider desirable. They afford a better bearing for cross-ties than does a top plate with its lines of rivets, and greatly simplify framing. So far as I have ever heard, they do their work well. I think $\frac{5}{16}$ preferable to $\frac{3}{8}$ inch as a minimum limit for the thickness of webs in plate girders. The latter limit frequently necessitates considerable waste in material.

The author specifies that all truss bridges are to be cambered with a rise of not less than $\frac{1}{1200}$ of their length. I am in doubt whether he means permanent or figured camber; but, be this as it may, it appears to me that a rule based on the length of span only is defective. The amount of camber evidently should be equal to the deflection under a full load, plus set, that the chords may be straight and square bearing when subjected to a maximum stress. The general formula for the deflection of beams supported at the ends and uniformly loaded, when ap-

plied to open web-girders, may be reduced approximately to the form $d = \frac{l^2 s}{C h}$, in which

d = deflection in inches due to dead and live load.

l = length of span in feet.

h = depth of truss in feet.

s = mean stress per square inch in chords in tons of 2 000 pounds.

C = a constant.

Mohr's formula for deflection is as follows: $d = \frac{e u s l}{E}$, in which

u = stress in member due to 1 pound weight at center of bridge.

s = stress per square inch in member when bridge is fully loaded.

l = length of member.

E = modulus of elasticity of member.

This formula is supposed to give reliable results, and some actual recorded deflections certainly agree very closely with it. It is, however, troublesome to work, or requires more time than a busy bridge-engineer is often able to devote to it.

The formula first given is simple and easily worked. The value of C should be determined by comparing calculated and observed deflections, but for the want of a sufficient number of recorded examples it has been obtained by Mohr's formula in the following cases:

Name of Bridge	Truss.	Mat'l.	l	h	d	$\frac{l}{h}$	$\frac{l}{d}$	s	C
Little River.....	Pratt.....	Iron....	99	24.5	0.882	4.0	1 347	4.20	1 904
Est.............	"	"	150	26.0	1.578	5.8	1 141	4.36	2 392
Tar River......	Whipple .	"	196	32.5	1.801	6.0	1 306	4.43	2 906
Henderson	Warren...	"	246	31.0	2.427	7.9	1 205	4.15	3 333
Susquehanna ...	Whipple..	Steel...	376	50 0	5.931	7.5	761	6.95	3 313
" ...	" ..	"	515	65.0	8.592	7.9	719	6.96	3 306
Henderson	Warren...	"	522	56.0	9.826	9.3	638	6.73	3 331

It will be observed that the value of C is nearly constant for spans between 246 and 522 feet, but falls off quite rapidly and uniformly for spans under 246 feet. Why it should do this I will not undertake to say, but assuming Mohr's formula to be correct, and allowing 10 per

cent. for set, practically the same results may be obtained by formula $d = \frac{l^2 s}{C h}$ by substituting for C the following values:

Spans up to 250 feet, $C = 900 + 8.4$ span.
" over 250 feet, $C = 3\,000$.

As this formula considers all the conditions, it is thought to be an improvement on the author's rule.

The author makes no greater provision for wind bracing than is made by many other engineers and much less than is made by some, nevertheless I have reason to believe that much material and money is being wasted in this direction. Up to about ten years ago it was not customary for bridge-builders in this country to figure wind bracing at all, but rods were put in of such size as was thought to be sufficient, ranging from 1-inch diameter, for spans of 150 feet and under, to 2-inches diameter for spans of 400 feet; or, say, about one-third average present requirements. These rods were frequently connected with the trusses in an insecure manner, usually at a considerable distance above or below the pin, developing large eccentric stresses, and often were ineffective, having nothing apparently to pull against, and these latter defects are also frequently found in much later designs. Thousands of these old and defective bridges are standing to-day; many have been taken down or strengthened on account of increased rolling loads; but very few have been blown down; and I know of no well authenticated instance of failure on account of insufficient strength of the lateral rods. Howe bridges have been blown down more frequently, but when we consider that they rarely ever have any portal bracing, or any provision for holding them up except the resistance at the joints of the lateral systems, which would hardly be considered in calculation; and the resistance of each truss to overturning about its own base, which does not cut a very large figure in calculation, it is not surprising that they occasionally topple over; this, however, is the exception, even when inclosed, exposing a full broadside to the action of the wind. Probably not one bridge in ten thousand is ever exposed to one-half of the force of wind that the specification considers. If the same rules were applied to buildings that are undertaken to be applied to bridges, but few could afford to build, and no contractor who would truly design and compete for such a building could hope to get a contract. The engineer who has had occasion to examine the exceedingly flimsy char-

acter of the great majority of iron buildings designed for mills and factories throughout the country, will readily admit that they are not in a condition to safely withstand any great amount of wind.

These insecure buildings, except in rare cases, appear to stand year after year without apparently any well-defined reason why they should, and it can hardly be claimed that human life is any more endangered by an insecure bridge than by an insecure mill in which hundreds of men are employed. These remarks are not offered as an apology for light and insecure work, for I would not be guilty of constructing it, and would freely condemn it wherever seen, but to call attention to what I believe to be a fact, that the danger to a bridge from wind pressure is much over-estimated.

Having intimated that the old bridges cited would fail under a wind pressure much less than that specified, it remains to prove that modern bridges would share the same fate notwithstanding the superfluous strength of the lateral systems. We will take as an average case the following example of a through bridge, Whipple truss:

Span, 201 feet	10¾	inches	center	to center of end	pins, 11	panels.
Depth, 32 "	0	"	"	"	chords, 20	feet clear.
Width, 17 "	9	"	"	"	trusses, 16	"

Dead load, 1 400 pounds per lineal foot.

Rolling load = { 2–80.6-ton consolidation engines followed by 2 240 pounds per lineal foot.

Bridges unloaded, wind 50 pounds per square foot, as per specification.
Horizontal force at top of end brace 22 340 pounds.
Bending moment on end brace 270 314 pounds.

The fiber stress on end braces at bottom portal strut will be as follows:

From portal bracing.......	20 800	pounds	per square	inch.
" overturning tendency,	1 240	"	"	"
" dead load............	1 980	"	"	"
Total............	24 020	"	"	"

24 020 ÷ 6 110 = 3.93 times the allowed stress for vertical loading.

The following table shows the condition of the bottom chord of the windward truss.

Panel.	Tension.	Compression.				Force per Square Foot Required to Collapse.
	Dead Load.	From Bottom Laterals.	From Overturning Tendency.	Total.	Resulting Stress.	
First....	36 800	38 100	23 100	61 200	24 400	30.0
Second..	36 800	68 600	23 100	91 700	54 900	20.1
Third...	52 900	91 500	23 100	114 600	61 700	23.2
Fourth..	79 700	106 700	23 100	129 800	50 100	30.7
Fifth....	97 100	114 300	23 100	137 400	40 300	35.3
Center ..	106 500	114 300	23 100	137 400	30 900	38.7

It appears to me inconsistent to use for the wind bracing a unit stress of but 1½ times as large as for vertical loading, when stresses 3.93 as large are permitted in the end braces; also to consider a force of 50 pounds per square foot on the lateral systems when a force of 20 pounds per square foot will collapse the structure. If the track stringers are spread wide apart they will materially assist the chords in resisting reverse stresses, but when placed under or near the rails, as they frequently are, but little reliance can be placed on them. A consideration of the leeward truss, when the bridge is fully loaded, taking into account the overturning tendency due to bridge and train, would show that the bottom chord of that truss was much overstrained. Although the above showing is bad enough, it would be much worse for a deck bridge, as the distance between trusses would be less and the surface exposed to the wind at top much greater than in the case considered. It is true that the author provided for these contingencies in his specification, but they are not provided for in practice, nor can they be until the present style of bridge-building is materially changed.

It is not probable that a train will be running at a maximum speed when exposed to a maximum wind, or when the wind alone is sufficient to upset an ordinary passenger car; for this reason I do not consider it necessary to add to the strength of the lateral systems on account of centrifugal force.

In all cases where the rods have adjustment, five tons are allowed for initial tension. This adds quite materially to the cost of a bridge, and does not appear to me necessary. Wind stresses and initial stresses are opposed to each other. The bottom lateral-rods of a bridge are not adjusted until after the span is swung. The bottom chords are pulled

straight by virtue of the tension in them, and in adjusting the rods there is no good reason why those crossing each other in the same panel should not have about equal tension; and the initial tension on one rod is held in by the rod in the opposite direction. When the wind stress comes on it is evident that its first effect is to relieve the rod opposed to the transmission of shearing stress to the supports, and when the wind stress exceeds the initial stress the latter disappears. The top-chord sections are faced squarely, and their natural position is straight, and a less force is required to hold them in this position than in any other.

If it is found necessary to screw up the rods too tightly during the process of lining up, if not found relieved, they should be relieved before final adjustment; but as the top chord is less flexible than the bottom, it is quite probable that some rods will have more stress than the corresponding ones in the opposite direction; but in this case, only the difference in initial stress can be added to the wind stress. So far for all practical purposes I do not think it necessary to make any allowance for initial stress, except when in excess of wind stress.

The specification requires that eye-bars when tested to destruction shall always break through the body. I do not think any manufacturer has yet succeeded in making iron eye-bars that will invariably do this, and so long as the bars have the requisite strength, it appears to me a matter of little consequence where they break.

With the exceptions noted, I have no objections to offer to the author's specification, and it certainly contains many good provisions.

GEO. H. PEGRAM, M. Am. Soc. C. E.—A great deal may be said on Mr. Wilson's very interesting paper. No subject is more fruitful of discussion, and it is fair to expect that much difference of opinion will be developed, and to predict a long step in advance in this most important matter of bridge specifications.

In keeping with the thoroughness with which Mr. Wilson has treated the subject, I have confined myself to the question of live load, in which, I think, some improvement can be made.

The custom of using two engines, followed by a train of uniform weight per foot, has been quite universal for some time. Mr. Wilson now, very properly, proposes three types of engines, because either may produce the maximum strain, according to the span and panel lengths.

But the amount of work necessary to produce the strain sheet is greatly increased, and where a bidder is required to make a dozen bids to each contract that he secures, all accompanied by complete strain sheets, the question of calculation becomes a serious one.

The direct use of the engine-loading in any case involves a deal of work, is often inexact, and, it seems to me, rather unscientific.

Some years ago the usual loading was a uniform weight per foot of span for chord strains, and the same load headed by one or two engine excesses for web members. The uniform load was sometimes varied, according to the length of span, but the variation was rather arbitrary and could not well be made to include the effects of driving-wheel concentrations of engines; the resort to the engine-loading would seem very natural and justifiable, but it is in some respects very objectionable. The labor of calculation could not be made greater. If the results represented the maxima for other engines, or for the same engines in other positions (head to head for instance), there might be some satisfaction, but they do not. To illustrate: The centers of gravity of two engines, as commonly specified, are about 50 feet apart, consequently for 25-foot panels the maxima concentrations will occur at alternate panel points, and therefore on the same web system of a Whipple truss; but if the panels are 17 feet, the maximum concentrations will be three panels apart, and therefore on different systems, and will not secure the maxima strains. Another evil to which engine-loading subjects us, is the variety of engines used, differing so little as not to materially affect results, but still necessitating special calculations. I believe the assumed loading should be rather excessive. In that case there would be less reason for such variety. The difference in weights of bridges proportioned for the lightest and heaviest loading now specified is only about 10 per cent., and with the train weights constantly increasing, a little excess strength should not be objectionable.

The nearest approach to a uniform load will best approximate the various engines and panel lengths, and afford the greatest facility of calculation.

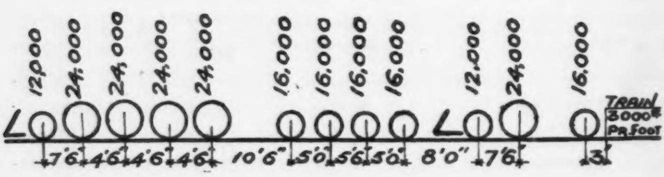
This loading should be modified to produce the effects of engine concentrations, and the actual engine loads should be the basis.

As fulfilling these conditions, I would submit the following specification as an equivalent of the loadings given in Mr. Wilson's paper, viz.:

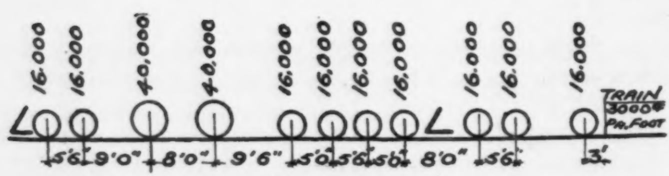
The live load on each track to be assumed in calculation, shall consist of a uniform load of 2 900 pounds per foot, together with a concentrated load of 25 000 pounds, the two to be combined in a way to give the maxima strains in all parts of the bridge.

A concentrated load passing over the length of the span is equivalent to a uniformly distributed load of double the amount; consequently, girders and the chords of trusses would be calculated for a uniform load of $2\,900 + \frac{50\,000}{S}$ (where S = the span) per foot of track. The web strains in trusses would be determined by adding 25 000 pounds to the common panel load at the end of the uniform load. This, of course, presumes that the uniform load shall extend over the panel in front of the concentrated load, and that the half-panel load at the advance panel point shall be neglected similar to present custom.

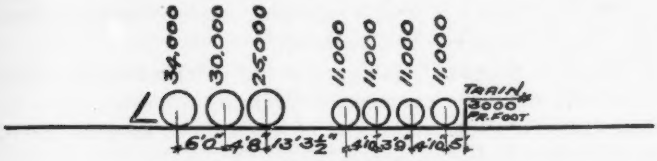
The following diagrams, showing the loadings taken from Mr. Wilson's paper, and the proposed loading, are introduced here for ready reference.



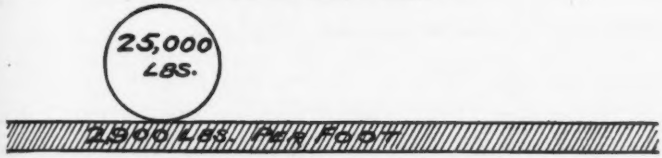
Typical Consolidation Engine Loading.



Typical Passenger Engine Loading.



Class "M" Engine Loading.



Proposed Loading (as equivalent of the above).

The following table gives the equivalent uniform loads for the several engines, obtained by moving the engines on the span to secure the maximum moment.

EQUIVALENT UNIFORM LOAD PER FOOT OF TRACK FOR GIRDER BRIDGES.

Span in Feet.	Engine.	For Middle Moment.	For Moment 10 Feet from End.	$2\,900 + \frac{50\,000}{S}$
8	Passenger	10 000	......	9 150
8	Mogul...............	8 500	......	9 150
8	Consolidation........	6 000	......	9 150
10	Passenger	8 000	......	7 900
10	Mogul	6 480	......	7 900
10	Consolidation........	5 280	......	7 900
12	Passenger	6 667	......	7 070
12	Mogul...............	5 667	......	7 070
12	Consolidation........	6 000	......	7 070
15	Passenger	5 333	......	6 233
15	Mogul...............	5 933	......	6 233
15	Consolidation........	5 760	......	6 233
18	Mogul...............	5 670	......	5 677
18	Consolidation........	5 333	......	5 677
20	Mogul...............	5 460	......	5 400
20	Consolidation........	5 280	......	5 400
30	Mogul...............	4 410	......	4 567
30	Consolidation........	4 480	4 560	4 567
40	"	4 070	4 134	4 150
50	"	3 712	3 940	3 900
60	"	3 547	3 760	3 733
70	"	3 438	3 633	3 615
80	"	3 370	3 543	3 545

The 8-foot span is so exceptionally short, that it is felt that the difference of 8½ per cent. may be disregarded.

In trusses, the panel length exercises such an influence on the strains that it was thought best to work out complete strain sheets. This has been done for a sufficient variety of conditions to show the general application of the loading proposed.

The class M loading was found so much below the others in the cases considered, that its results will not be given.

The Pratt (single intersection) and Whipple (double intersection) trusses were taken.

The strains are given for chords, end-posts and ties. For convenience of reference, the two end-chord panels (in which the strain is the same) are No. 1, the third panel 2, etc. The end-post is 0, the hip-suspender 1, the first inclined tie 2, etc.

The dead load is included in order to give total strains. It will be understood that the loading given is for one track, while the strains are for one truss, and therefore due to half the load.

CASE 1.—Pratt. 150-foot span in 9 panels of 16 feet 8 inches. Depth, 25 feet. Total dead load, 216,000 pounds.

	Chord Strains.			Web Strains.		
	Passenger.	Consolidat'n	Proposed.	Passenger.	Consolidat'n	Proposed.
0	......	......	......	184 100	187 800	186 930
1	106 400	104 400	103 824	45 700	48 500	46 666
2	181 640	176 400	181 692	146 280	145 600	145 176
3	229 540	225 800	233 604	106 800	106 600	106 542
4	258 100	253 100	259 560	70 600	70 700	71 130
5	......	......	......	37 700	37 900	38 940
6	......	......	......	8 400	8 900	9 972

CASE 2.—Pratt. 150-foot span in 6 panels of 25 feet. Depth, 25 feet. Dead load, 216 000 pounds.

	Chord Strains.		Web Strains.	
	Consolidation.	Proposed.	Consolidation.	Proposed.
0	......	......	204 600	204 510
1	146 000	146 062	64 200	66 700
2	228 000	233 699	133 000	134 026
3	257 400	262 911	70 600	72 100
4	......	......	16 600	18 607

CASE 3.—Whipple. 255-foot 6 inch span in 14 panels of 18 feet 3 inches. Depth, 29 feet. Total dead load, 591 000 pounds.

	Chord Strains.			Web Strains.		
	Passenger.	Consolidat'n	Proposed.	Passenger.	Consolidat'n	Proposed.
0	......	......	......	382 080	381 100	378 506
1	204 000	203 500	202 141	56 900	59 100	60 062
2	297 700	297 600	205 414	178 500	177 000	182 127
3	446 800	442 000	450 869	204 100	202 400	208 768
4	575 800	572 500	575 233	170 800	169 400	175 132
5	667 200	662 100	668 506	134 300	133 300	138 450
6	724 500	719 400	730 688	104 200	103 500	107 855
7	754 500	751 100	761 779	71 000	70 700	74 216
8	......	......	......	43 800	43 800	46 664
9	......	......	......	13 200	14 000	16 067

CASE 4.—Whipple. 255-foot span in 10 panels of 25 feet 6 inches. Depth, 29 feet. Total dead load = 600 000 pounds.

	CHORD STRAINS.		WEB STRAINS.	
	Consolidation.	Proposed.	Consolidation.	Proposed.
0	....	......	419 900	415 807
1	277 100	275 116	76 200	79 475
2	392 900	397 390	199 100	193 899
3	581 400	580 801	229 200	224 242
4	701 000	703 075	172 700	169 110
5	756 600	764 212	108 500	106 409
6	....	....	59 400	58 646
7	....	....	2 400	3 415

*CASE 5.—Whipple, 403-foot 9 inch-span, in 19 panels of 21 feet 3 inches. Depth, 42 feet. Total dead-load = 1 560 000 pounds.

	CHORD STRAINS.		WEB STRAINS.	
	Consolidation.	Proposed.	Consolidation.	Proposed.
0	....	....	744 464	736 464
1	336 400	333 100	81 900	83 500
2	493 400	491 800	351 300	351 700
3	769 000	767 600	402 000	401 700
4	1 012 000	1 009 000	344 200	346 200
5	1 216 400	1 211 700	302 600	304 700
6	1 378 300	1 374 200	249 600	251 600
7	1 511 000	1 507 800	210 300	212 600
8	1 595 100	1 601 300	159 800	161 700
9	1 654 000	1 655 900	123 000	125 000
10	....	....	74 600	76 400
11	....	....	40 300	42 000

In Case 3 it will be observed that the tie strains are slightly less with the Consolidation than with the proposed loading, while in Case 4 they are greater by about the same amount. This is due to the manner in which the maxima engine concentrations strike different truss systems in one case and the same system in the other as previously explained.

* In calculating strains for uniform loads, it is not necessary to separate the systems except when the number of panels is odd.

The proposed loading is submitted simply as an equivalent to those given in the paper under discussion, and to illustrate the fact that a variety of engine and train loads may be represented by a uniform load in combination with a single concentrated load.

As a standard loading, I would suggest making the uniform load 3 000 pounds, and the concentrated load 30 000 pounds. This will cover an increase in the engine-loading which will probably take place at no distant day.

The Baldwin Locomotive Works build a Decapod locomotive, as it is called, which is heavier than any given in the paper. It has five pairs of driving wheels on a base of 17 feet, with 25 600 pounds on each pair, 16 000 pounds on the pair of truck-wheels, and 20,000 pounds on each of the four pairs of tender-wheels.

This engine is designed for mountain service, and may not come into general use. These engines would produce slightly greater strains than the suggested loading, but not enough, it is thought, to be taken into account until their use becomes very general. The difference would be greatest in some girder spans where the customary percentage added for impact would be excessive on account of their slow speed, and thus tend to equate the loads.

C. C. Schneider, M. Am. Soc. C. E.—I have read Mr. Wilson's paper with great interest, and consider it a valuable contribution to the profession.

Mr. Wilson has departed from the usual specifications, by using a modification of Weyrauch's formula for reducing the permissible unit stresses, and thus increasing the sections for impact. I have of later years used a similar formula for the same object. Not that I believe in the correctness of the formulas of Launhardt and Weyrauch as deducted from Wöhler's experiments, but as a convenient empirical method of increasing the sections for impact in proportion of the ratio of dead to live load.

There are a few points, however, in Mr. Wilson's specifications, on which I hold different views.

Like most other specifications, Mr. Wilson's gives three formulas; or rather, one formula with three different coefficients in the denominator for compression members, distinguishing fixed ends and hinged ends.

I have for several years past used only one formula for all compres-

sion members, regardless of their end connections, employing the one which is on the side of safety, viz., that which applies to struts with hinged ends.

A compression member in a structure is not in the same condition as in a testing machine; and while in a testing machine the struts with fixed ends show a greater strength than those with hinged ends, it is different when the struts are in a structure. The strut with the ends fixed in a testing machine is carefully adjusted, with the surfaces pressing against its ends exactly parallel and at right angles to the axis of the strut; while in practice, the strut with fixed ends receives its pressure mostly eccentric, and a bending strain at the same time. The conditions of the strut with hinged ends in the structure come nearer to those which prevail when the strut is in a testing machine than is the case with the strut whose ends are fixed. The strut with hinged ends will receive its direct pressure practically concentric in the structure as well as in the testing machine, and the bending strains, originating with the friction in the pins, must necessarily be considerably smaller than in struts with fixed ends.

I am therefore of the opinion that if any discrimination is made between struts of different end connections, it should be in favor of the hinged ends.

I perceive that this opinion is also held by other engineers. Mr. Theodore Cooper, in his latest specifications for highway bridges, uses only one co-efficient in the formula for compression members, regardless of their end connections.

According to my experience, this discrimination in favor of struts with fixed ends has done a great deal to encourage contractors to design details which must be considered bad practice, they taking advantage of the specifications for the purpose of saving material.

Mr. Wilson considers compression members which are continuous over points of support as hinged; this I consider a move in the right direction.

However, I fail to see the reason why Mr. Wilson follows the older specifications, and specifies a smaller unit strain for compression than for tension.

All experiments prove that the ultimate strength, as well as the elastic limit, are practically the same for compression as for tension; why then not allow the same working strain in both cases?

The usual explanation given by engineers who adhere to this practice is: The material in compression members buckles or cripples before rupture takes place; therefore, for compression members we take this buckling strength as a guide to determine the permissible unit stress, while for tension members we take the ultimate strength. A good rule, however, should work both ways. The tension members are also crippled before rupture takes place; and in this respect all members are alike. Compression, as well as tension members, are practically crippled as soon as their fiber strains exceed the elastic limit of the material. I think we should be guided by the elastic limit of the material in determining the permissible stresses more than by the ultimate or buckling strength; as in practice we do not intend to reach either, but we want to keep enough below the elastic limit to leave a fair margin for defects in the material and for ignorance.

I would also call attention to the following clause in Mr. Wilson's specification: "In all cases where the rods have adjustment, an addition to the above stresses of five tons must be made for initial tension." This appears to be inconsistent with the formula previously given; as the initial tension does not affect the maximum stress, but increases only the minimum stress on that member, and hence I can see no good reason for increasing the section on account of this initial strain.

Mr. Wilson allows a variation between the diameter of pin and pin-hole of one thirty-second of an inch; this, in my opinion, is more than should be allowed in good practice. I think the variation between diameter of pin and pin-hole should be limited to one-fiftieth of an inch.

Of the various existing specifications, the "Erie" specifications may be mentioned as among the most prominent ones. They were in existence previous to most of those enumerated by Mr. Wilson, have been used as the standard specifications by many railroads, and have been extensively copied by engineers.

T. C. Clarke, M. Am. Soc. C. E.—Mr. Wilson has aimed in this paper to present a specification which shall insure good results. He makes one grave mistake, which has been pointed out, but which it will do no harm to call to his attention again. He prescribes methods of manufacture, and then demands certain results. These two things are incompatible. Either can be insisted on, but not both.

He makes no mention of steel as a material of construction. This,

perhaps, adds to the dignity of his paper, but deprives it of much of its present value.

I am prepared to state that the following things can be demonstrated to be true:

First.—Steel can now be obtained of a more uniform and reliable quality for bridge construction than iron.

Second.—Bessemer steel can be obtained of as uniform and reliable quality as open-hearth steel, provided the ultimate strength does not exceed 70 000 pounds per square inch.

During the last fifteen years, the principal improvements in bridge construction are as follows:

I.—Use of better materials.

1. Substitution of rolled iron for cast-iron.
2. Substitution of steel for rolled iron.
3. More thorough system of inspection.

II.—Increasing the strength of different parts of a bridge, in direct ratio to the proportions of live to dead load, and to the direct action of live load upon them.

III.—Providing for wind pressure.

IV.—Better connections.

1. Better proportions of eye-bar heads and pins.
2. Use of pin joints at top and bottom of end posts.
3. Better attachment of floor system to trusses.

V.—Improvements in design.

1. Less depth of truss.
2. The general use of long panels.

Many other things will doubtless occur to readers of this paper.

Jos. M. Wilson, M. Am. Soc. C. E.—Mr. Cooper is somewhat in error in assuming that the paper in question was presented for the sole purpose of obtaining the benefit of criticism from bridge experts. While criticisms and discussions are very desirable, and are welcomed by me as tending to the ultimate of a standard specification for the American bridge engineer, yet my particular object was to offer the paper as a contribution to the profession, supposing that my long practical and successful experience, which is hardly as limited as one might be led to infer from Mr. Cooper's remarks, would give a reliability to my methods valuable to others who might be traveling in the same

direction. The specifications submitted are free for the use of any who may desire to adopt them, either as they stand or with whatever modification or improvement they may deem advisable, and I trust that they may be of some service to my fellow-laborers.

I adopted the formulas as advocated by Launhardt and Weyrauch, not because I believed them perfect or fully covering the case, but because, in my opinion, they were an approach to a more rational and proper method of considering the subject than anything that had as yet been brought to my notice. I believe that the true method in forming a specification of this kind is to generalize it as far as possible. I am glad to notice that the interesting nature of Wöhler's experiments is admitted by Mr. Cooper, and their correspondence with the fatigue to which material is subjected in actual practice. It is true they are somewhat limited in extent, and it is to be regretted that there are not more of them; but, so far as they go, they lead us to what we want, and I fail to see where Mr. Cooper proposes anything better or more reliable. If his great practical experience or numerous experiments have produced anything better, I have not as yet had it brought to my notice. The results of the formulas as given by me certainly compare very favorably with the practice of high authorities, varying generally on the side of safety, and they are undoubtedly more rational than the haphazard method so generally adopted. Plate No. XXXVII shows that the results are in harmony with the practice of different engineers by other methods, hardly any two of which agree. And as to formulas, each engineer has his own, and very little basis on which to found it. Take the formulas, for instance, proposed for alternate stresses by Mr. Bouscaren in his criticism. He may be satisfied with it, and he admits that he adopts it from Launhardt, but what have we in the way of experiment to confirm it, except by comparing it with what others adopt, and what authority have they for their assumptions. If they have made extensive experiments, we have not heard of them. All are empirical, and my effort has been to use a general expression that I thought would represent or come sufficiently near to the average of the best practice, and at the same time be surely within the limits of safety as known to us. I am confident that any comparison between the results by my method and any of the old methods, as advocated by several of my critics, will show to the advantage of my formulas. The remarks of Mr. Mace Moulton tend to confirm this. It is true that I have raised the working stresses

for compression; but this, I believe, is more in unison with the latest experiments, and many favor raising them still higher.

I admit the desirability of having regard to the elastic conditions of the material rather than to the ultimate strength, and I believe that my specification shows it; I cannot allow Mr. Cooper's objection concerning the use of these formulas, that anything is left in indefinite shape, or that varying interpretations can be made by different designers. The method, on the contrary, is rigid and exact, and if properly followed, must result in compelling a just and fair competition, and first-class work. If any one bidding under a specification fails to conform with it in every particular, it is the duty of the engineer receiving the bids to point it out and to insist upon a compliance; a matter of no difficulty if he understands his business and conscientiously follows it. I speak from considerable experience on this subject. Bidders can be compelled under this specification to proportion the work exactly as it is wanted, and there are no two or three interpretations to be followed. It will never be possible to prepare a specification on which an engineer, ignorant of the laws of bridge construction, can obtain true bids without expert advice. The plans should always be submitted to an acknowledged expert before adoption, and the work should be carried out under his direction.

I presume, as a fundamental starting point, that when bids on a bridge are received, the plans, details, etc., are examined by a competent and reliable engineer, who not only has authority to compel compliance with the specifications, but can also require proper and approved forms of detail.

My experience may, perhaps, have been somewhat limited. I can only say that it has been connected with actual bridge work for over twenty years, during which time more than 20 miles of bridges have been constructed from my plans and specifications. Most of my bridges, it is true, have been built from my own designs, but not all of them, and I have had to deal with contractors who were quite as ready to take advantage of opportunities as any. In these dealings I was entirely satisfied with the completeness of the specifications and their ability to cover the case.

In regard to impact, I am aware that we do not have very much data to draw from, but as far as the law has been known, I think it has been met. Prof. Robinson may throw some light on the subject, but if Mr.

Cooper, as he says, does "not know with sufficient definiteness the effects of impact upon a structure and its individual members to formulate any law," how can he adopt a rule-of-thumb method which will meet the case. We know at least that impact is a function of the live load; why not then treat it as such and let its effects vary accordingly? This is surely better than guessing at it for each individual member of the truss. The form and construction of any member certainly has a large influence upon its ability to resist stress of any kind, impact or otherwise, and it is best, that after we decide what stresses come upon these members, we should consider this question of form and also the mode of construction of the member, the number and disposition of its parts, rivets, etc., all of which is taken care of in its proper place.

The reason why girders formed with web plates and angles only, having no upper flange-plate proper, are not allowed, is largely a practical one. The upper web plate is a great protection against the weather, or the admission of water, charged, perhaps, with sulphurous or sulphuric acid, to the joints of the iron. In addition to this the plate stiffens the angles on opposite sides of the web very materially, making them act more thoroughly together.

In my criticism on Mr. Cooper's article "on the rules for pitch of rivets and thickness of web" in riveted plate girders, I merely give the example as an instance of how a strict rendering of Mr. Cooper's rule would operate. I did not mean to infer that Mr. Cooper would really place the stiffeners in that way, as I felt sure he would know better than to follow strictly his own rule.

In reply to Mr. W. Howard White's query as to the prohibition of allowance of web in calculating plate-girder flanges, I would say that I do include the web in solid rolled girders where the thickness is comparatively great, the whole substance of the cross-section in one homogeneous mass, and the amount of material generally in excess for shear; but in the case of built girders with the plate web riveted to the flanges, where the flanges are formed to take the horizontal stress, and the web the shear strains, I think it is better to keep each to its own particular duty without depending on the other, and it is practically on the side of safety to so consider it. According to the theory of some, this web already has to resist a tension in one direction and a compresion at right angles to it. To introduce a third force in a diagonal direction would seem almost too much, the argument, if extended, leading to a "*reductio*

ad absurdum" that a web plate can resist any number of forces of either tension or compression in any direction at the same time, provided they do not follow precisely the same lines, each force being up to the full capacity of the sectional resistance of the plate at right angles to it. It is only of late years that I have required in plate-girder work, flange plates in compression, where spliced, to be covered to their full section, as it has been found almost impossible to obtain truly butted connections. Broken joints in built compression members, as posts, etc., where planed and well butted, would only need plates sufficient for proper stiffness.

I see no reason for increasing the number of ties under the rails beyond what is necessary to furnish proper support for them, and 20 inches to centers is a convenient distance for spiking. In case of wheels getting off the track they will run over these 10-inch clear spaces without difficulty, as practical experience has proved it. The outside timber guard-rail, well notched to the ties, is considered necessary to prevent the ties from crowding together, as well as a guard. The inside guard-rail may be necessary in certain special cases, but generally it is an open question whether it does not increase the danger from catching brake blocks, etc., between it and the regular rail and running a train off.

As to the limit of pressure on masonry, I think 300 pounds per square inch (21.6 tons per square foot) is sufficient for ordinary work. I have provided for a larger amount being allowed in specially authorized cases where it is expedient. I think 21½ tons per square foot is about twice what it should be for fairly good brick-work. Thurston permits on brick-work in cement a load of 10 tons per square foot. Trautwine states that ordinary brick-work cracks with 20 to 30 tons; good brick-work in cement 30 to 40 tons; and that first-rate brickwork in cement will crush with 50 to 70 tons per square foot. One-sixth to one-tenth of these figures would be considerably below what Mr. White gives. Sandstones and ordinary good local building stones are very variable in crushing strength, and I do not think that the limit given in the specification is too small for general cases, particularly when the mason-work is not under the control of the bridge-engineer, as is often the case.

In regard to the use of paints, I did use red lead for some years as a priming coat at the works, and while I believe it to be of excellent quality for this purpose if properly put on, yet there was considerable difficulty in getting it applied. Humber, in one of his works on bridge

construction, very strongly recommends a special preparation of iron, and condemns all lead paints. Further information on the paint question is very much desired. An investigation into this matter in a chemical laboratory on a prominent railroad developed the fact, I believe, that paint made from the refuse of zinc batteries of the telegraph service was the proper thing to use, but every one knows that zinc paints will not stand the weather.

In reply to Mr. Burr's objection to my neglect of the load in advance of the drivers, I would say that I do not neglect the load ahead of the drivers; but in calculating web members I intend that the drivers shall be so placed as to give a maximum cross-girder load, and any load on the previous cross-girder is omitted, as I believe this will give maximum effects in the web members.

Concerning the matter of web stiffeners I of course admit, as I stated in my paper, that a web of sufficient thickness compared to its depth will resist by compression and tension just as Rankine computes it, but when the question of stiffeners comes to be considered, it is for webs very thin compared to their depth. In the model which I used, the web was of thin letter-paper with practically no stiffness in it at all. The stiffness which would be imparted to it by tension in the other direction is of no importance, as a deflection ruinous to the bridge would be reached before it could come into effect. Take a long, thin bar under tension. How much force is required at right angles to its length, applied at its center, to give it quite a sensible deflection ? Its own weight will do it. If we assume that the stiffeners are not fastened to the web and have no end bearing, but are merely held against it so as to prevent any motion in a direction perpendicular to its plane, we would then have the action as described by Prof. Rankine, in which the web would be considered as a fixed column in a diagonal direction between stiffeners, of a length equal to the diagonal, and the action as described by Mr. Burr would then take place. The computations, followed strictly according to Rankine, would deduce a thickness of web that would be out of all proportion to practice. To keep that thickness down to reasonable limits, Mr. Burr would have to place his stiffeners very much closer than practice requires. If, however, the stiffeners are riveted to the web and have end-bearing under the flanges, compression to the web cannot take place without a resistance from the stiffeners, the strain being transferred to them by the tension on the web, either

through the rivets or *via* the upper flange. This action may be more or less partial depending upon the thickness of the web, but to avoid uncertainty I consider it best to put sufficient section in the stiffeners to take it.

Consider a lattice girder with a number of intersections, the web members running in opposite directions, inclined at, say, 45 degrees with the vertical. Those taking tension are computed as such, those resisting compression are computed as struts. Now let the compression members be placed vertically and the tension members be left as they are. The same condition applies, remembering that some diagonal members must exist in both directions for tension from direct and counter stresses. Now develop the lattice into a plate-girder by making the tensive members of the web continuous, and by concentrating the compressive members into vertical struts at regular intervals apart, equal to, say, about the depth of the girder, we have the condition of a plate girder calculated as per my method. If the original lattice with its web members remaining at 45 degrees were developed into a plate-girder in the same way, the compression struts would be at 45 degrees instead of vertical, and if both tensive and compressive members were developed into plates, the condition would be that of a plate girder without struts, and would be calculated as per Rankine's method. The natural direction for the forces to go would be in the inclined direction, and, theoretically, it would be best to put the struts inclined, but they are placed vertically as a practical matter of facility in manufacture and in making of connections. A portion of the web held between the struts, and extending for a short distance outside of them, depending upon its thickness and the question of its liability to buckling, may act with the strut to take compression. I do not wish to be understood as denying that a plate will take compression and tension at right angles on the same section, but with deep webs it is safer not to go down too close in these matters.

I beg to thank Prof. Merriman for his beautiful exposition of a method of determining the working stresses for members subjected to varying stresses. I have not seen his method of deducing his formula, but it certainly appears to give very rational results.

I have read Prof. Robinson's criticisms and recommendations with great interest. His paper requires considerable study, and I am at present, for want of time, hardly prepared to say how it might influence

me in the matter of modifying my specification in the future. His first recommendation is one, however, not very difficult to provide for, and well worthy of consideration. There is some risk in lessening the allowance for safety until we do know all the causes which we have to provide against. We must be sure that we eliminate all the ignorance before we ignore the co-efficient of ignorance. It would be very satisfactory to know positively all the stresses to which a bridge may be exposed, and we would like to take advantage of any rules in arrangement or distribution of parts that would tend to lessen, for instance, the effects of impact. But practical experience with what we call our good bridges would still warn us that the results should not give us lighter ones, unless it can be proved that the arrangement of panels, or other means of avoiding certain actions of forces, is a positive improvement for general cases. A law for the lengths of panels might be adopted to meet certain standard wheel spaces, and bridges erected in conformity with it. Then, some time hereafter, it is more than likely that new designs of cars to suit particular requirements of trade would be put into service without any consultation or even notification to the engineering department, the proportions of which would eliminate all the advantages derived. Bad joints in the track would be a source of increase to the percentage for impact that could hardly be estimated. Experiments recently made in Europe by M. Considère in reference to increased stresses in members of bridges from passing trains, showed a marked increase in the stresses from bad joints in the rails. It seems to me also that the speed of trains would affect the amount of impact very materially, and that this would be a difficult force to take account of with any exactness, as, in the case of freight trains particularly, the speed is very irregular.

The effect of the unbalanced engine wheel, as deduced by Prof. Robinson from his experiments, showing, as it does, a material increase with the length of span, is so at variance with the generally accepted practice, that it will bear considerable examination and investigation before being adopted.

Very often when bridges break down, facilities for getting at the real cause, or of forming an opinion on the same, are not afforded by the railroad company, in order to avoid the responsibility that might ensue were the true cause known. I believe that in all cases of failure, if the full details of the structure and the loading can be obtained, it will be found that some part of the bridge was not designed in consonance with

the recognized practice of the present day among first-class engineers. The mystery with which the case is often shrouded is only a cover for the protection of the railroad company.

If we know the true amounts of all the forces to which a bridge can be subjected, and raise the unit strain accordingly, lessening the factor of safety, the result would, I think, not give us essentially lighter bridges as a class, although better designed and with a more uniform proportioning of parts, as we would have greater stresses to take account of than we know now. Of course the factor of ignorance would be lessened to a minimum, and this would be much more satisfactory. Our experience shows us that there is a large factor needed for wear and tear, and also a margin for the future exigencies of the service. We have never yet been able to anticipate the future sufficiently to allow our railroad bridges to last to anything like the limit of the durability of the iron. The material may last indefinitely, but in Europe, as well as in this country, bridges which have not worn out are constantly being replaced, solely because they are too light for the increased service.

Professor Robinson takes some exception to the values in the formulas of Launhardt, Weyrauch, etc., which would open up a discussion, going behind my assumption, and raising a question as to the correctness of the deductions of eminent authorities, which I will not now attempt to consider. Such a discussion would be very interesting if it could be followed out, and might result in modifications of value.

In reply to Professor Swain, I would say that, so far as I am prepared to say now, I would not like to increase the stress for iron under quiescent loads to a greater amount than 15 000 pounds per square inch. Professor Cain's method of allowing for impact appears to me to give results in consonance with practice, and it is very convenient of application. Professor Robinson's researches are throwing a good deal of light on the impact question, and I am hardly prepared to say just now what ultimate view I may take of the matter. In regard to the formula of Winkler quoted by Professor Swain, a comparison with my formulas using the same limiting stresses for all live and all dead-load as I do, would give results for intermediate cases not varying very essentially from mine. But with the values Professor Swain suggests for American practice, the results would be considerably above what I would like to adopt.

Professor Swain's suggestion for calculating the upper chord when

subjected to cross-strain is not tenable, as the chord does not act as a supported beam, and it must be taken in the condition in which it actually exists. The moments may be positive or negative according to the loading. Some approximation may be used in obtaining the required values, but we should endeavor to obtain the maximum effects.

Mr. Moulton's deductions and comparisons are interesting as showing the correspondence between the formulas of the specification and practical experience. The question of raising the value of the working stress in compression to that used in tension is a matter that will bear consideration, but I do not think we have sufficient data as yet to fully justify this departure, although I can signify my inclination in that direction.

Replying to Mr. Boller's criticism concerning continuous compression members being hinged over points of support, I think the same principles would apply as in continuous columns stiffened at intermediate points. A pin, while it would hold the column in place, would give no bending resistance around an axis through its center. If the chord is so loaded that adjoining bays tend to bend in the same direction, then the assumption does not apply, and in the case of an upper chord of a through bridge, the moment due to the weight of the member itself would probably have this effect. But for deck bridges with a heavy load on one bay and only the dead weight of the member itself in the next, the downward moment of the latter is easily overcome by the reflex moment for the adjacent panel.

Concerning the criticism of Mr. William Sellers in reference to the use of double-rolled iron and the exclusion of scrap-iron, I would say that, as a general rule, I am myself opposed to specifying methods of manufacture at the same time that certain conformity to physical tests is required, and some twenty years ago modified the specification then used on the Pennsylvania Railroad very materially in this respect; but the requirements of double-rolling and omission of scrap was retained at that time, because I believed that in the then state of the manufacture we obtained a better quality and more uniform material than we could otherwise do. These restrictions have been retained ever since, although I am willing to acknowledge the arguments on the other side, that some iron is better not double-rolled; that the manufacture has progressed so that there is no difficulty in meeting the specification without this requirement; that scrap has to be added to some iron to obtain

the wished-for result; that the manufacturer in his own interest will not use short or improper scrap, the increased facilities for testing, etc., making it too much risk to do so; and also, above all, that it is extremely difficult, if not impossible, to know that we obtain what we specify, it being almost certain to be otherwise, to the detriment of conscientious bidders for material, etc. Bearing in mind also the greater advantages and reliability of the methods of testing employed of late years, I am perfectly willing to modify my views in this respect, particularly so as it will tend to promote harmony of action.

The proportions for test pieces adopted were believed to conform to those used by the United States Government Inspectors, and it was considered exceedingly desirable that there should be a standard in this respect. I admit the advisability of having the length a uniform multiple of the shortest transverse section, and would willingly approve of a common standard for test pieces if one can be agreed upon.

As to the limitation of the specification to wrought-iron alone, I would say that I have not the data at hand to take up the question of steel in the way I should like to treat it. My experience with steel has not been very great, and while I should not hesitate to use it for heavy compression pieces or large chord links, especially in a long span bridge, yet I hardly think I know enough about it to attempt to form a general specification for it.

Mr. Thatcher's suggestions in reference to the simplification of the work of computation are certainly worthy of consideration. I am uncertain as to whether a uniform load can always be found that would produce essentially the same results as the wheel loads, but I have no doubt that tables could be prepared covering the maximum effects for the several classes of engines, which would facilitate work. We have already tables in use which cover a great many of the calculations of the specifications.

Concerning the rule that the cross-girder load shall be considered as the head of the train in proportioning web-members, and Mr. Thacher's criticism that the percentage of gain on the different members varies largely, I would say that I do not think this variation amounts to more than the variation in the actual weights of different engines of the same type from the assumed weight for that type. There will always be a slight gain in stress in the member which is a maximum, but I think this will not exceed the variation for different engines.

As to the division of dead-load at panel points, I agree that an approximate general rule might be an improvement.

I quite agree with Mr. Thacher that it is poor construction to carry the floor of bridges on the upper chord between panel points, but it seems difficult to avoid it in many cases. The determination of the stresses from the two kinds of loading is undoubtedly involved in some uncertainty, but I have endeavored to cover the case in the method I have adopted. I have treated continuous chords only, and my intention is that the combined stress in the outer fibers shall not exceed the column stress b at the middle of the panel, or the compression stress a at the panel point. I found no reliable formula for giving the effects in a continuous chord due to the moments from pins being out of the neutral axis, and therefore had some investigations made which proved that while the moment from each pin affected every panel towards the middle of the span, the alternate panels having moments of different signs, yet the combined results amounted to very little except in the first and second panels and the pin point between them. I prefer, however, to avoid these moments as far as possible by keeping the pins very near the neutral axis, and for any slight effect in the first panel there is generally an excess of section over the theoretically required amount. I have considered the dead load as acting on a continuous beam, and the moment for live load as three-fourths that on a supported beam, believing it to be a close approximation to the effect from a single rolling load on a continuous beam. The section thus given for the middle of the panel is generally in excess of that required at the panel point, so that it was not necessary to specify that the value of a should not be exceeded in the lower flange at that point.

The question of excess of material in the heads of eye-bars is undergoing considerable modification from the old rules, as I intimated in the body of my paper. I am not prepared to say now what modification may be made, and probably no general rule would apply to all methods of manufacture. The essential point however is, that the head should be fully as strong as the body of the bar, and the less metal that can be used for this purpose the better.

The use of constant values for bearing stress and for bending stress of pins might give less labor in making calculations, but I do not think this of sufficient importance to allow us to ignore the differences which would then occur with extreme values of the unit stress.

The limiting distance of twelve times the thickness of the plate between centers of rivets is based on the rule that the compressive strain in the flange is limited to that for a length of twelve times its width.

I do not believe in having the tie plates at the ends of compression members too long, but I think it well to keep up the number of rivets as the specifiation requires. Future experiments may cause some modification of my views in this respect.

In the special cases of standard girders mentioned by Mr. Thacher, I would say that where rivets are staggered in rows near together, I would not require those of each row to be kept twelve times the thickness of the plate apart. I think that compression flanges should always be well riveted together, and I object to separate plates or pieces unless they are so riveted as to allow no tendency to separate action. The 44-foot standard girder mentioned, was designed some years ago, before the present specifications were prepared, and does not conform strictly to them in their present shape, as some modifications have been introduced.

The matter of camber is to my mind a question of appearance only, and generally I have the camber framed out in the track so as to make it level. I do not like to see a bridge fall below a level line and the specication gives a minimum rise only. The rule of the specification is, I think, sufficient for practical purposes.

I do not make any claim to originality in the formula for wind bracing, having adopted that which seemed best to suit my views. My experience with old bridges, however, has shown many cases in which the bracing was too light for ordinary use without any extraordinary wind action, in many cases it being impossible to keep the requisite lateral stiffness in the structure, the rods stretching continually under ordinary traffic from the action of other forces than wind. The assumed wind force is taken as a maximum, and of course it is not as severe in some localities as others, but it would be hardly possible to make any distinction for location, unless it were for a place where very extraordinary precautions would have to be taken.

Mr. Pegram's discussion on live load really proposes a more expeditious way of arriving at the same results as the specification requires. In the old days of my labors in the profession, I used, in common with other engineers, a load per foot lineal of each track to cover all cases. This method was improved upon by taking account of the engine wheel-loads. It was found, however, that no sooner was the latest and

heaviest engine in use on the road adopted as a standard, than a still heavier engine was brought forward. To provide for this, typical engines were assumed, which were supposed to cover what would be the actual increase in weights of motive service for some years to come. These typical engines had been in use for only a few years, when the third engine of the specification was brought into service, and it was found that some of its effects on cross-girders and for short spans were in excess of those from the typical engines. I therefore added it to the list. Its effect is limited, and only comes into play in a few cases. I have no doubt that half a dozen types of engine could now be found which exceed our typical types, although, perhaps, not much. This only shows how inexpedient it would be, even with more scientific and exact methods of calculation, to lighten our bridges. The great point in favor of stone bridges, and really their saving clause, has been their great excess of strength. Those who favor stone bridges say: Oh yes, they cost more money than iron bridges, but they are so much more permanent. I say, put the same excess of material in your iron bridges, or rather the same excess of cost, and you will have a structure much better adapted to resist travel, time, and the elements, than you have in a stone bridge. I have never been in favor of stone bridges of any size for railroad traffic, particularly in our climate. The material, from its brittle, crumbling nature, is very ill-adapted to resist the action of variable loads, impact, etc., and wherever frost can enter, it slowly but surely does its work. I have examined arch culverts over many miles of road; I have watched carefully some stone spans of considerable size, and have had to do with extensive repairs. I know that this is so, and the dilapidated condition of many even large culverts on some of the roads, is a disgrace to them. It is fortunate that they are below and out of sight.

To come back to Mr. Pegram's discussion, however, there may be some difficulty in arriving at an entirely satisfactory solution of the maximum effects by a simple system, but I believe it to be well worthy of the attempt. In using one set of engines similar to those in my specification, it is easy to make out equivalent uniform loads, giving the same moments as the wheel loads, and also the panel loads for all lengths of panels. Therefore, I did not appreciate the necessity of simplification so much as manufacturers would do, who have to deal with a variety of specifications. Any system covering the maximum

effects would have to be somewhat in excess in some members. I am hardly prepared to say how far Mr. Pegram's loading will satisfy the case. This will require some investigation.

In reply to Mr. Schneider, I would say that it is very seldom columns in bridges are other than hinged, but the specifications were intended to be general and to include all cases. Sometimes fixed ends do occur; but when doubtful, I should certainly consider the columns as hinged. I do not quite agree with Mr. Schneider on the question of initial tension as not affecting the maximum stresses in counter rods. We know from practical experience that when counters are screwed too tight in a bridge, the loading upon them will break them, and evidence of this often occurs. The initial strain screwed into the rod while the bridge is at rest unloaded, it seems to me will remain there, in addition to the counter strain that is put upon it.

When Mr. Clarke states that I make a grave mistake, I presume he refers to the question of double-rolled iron and the omission of scrap, a matter to which I have already replied. I am hardly prepared, however, to admit the great gravity of this mistake, although the requirement may be inexpedient.

In conclusion, I beg to thank those who have taken part in the discussion for the attention they have kindly given to the matter, and for the many valuable suggestions they have made. I trust that the result will ultimately tend towards truer methods of bridge calculation, and to greater uniformity in specifications.